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The Association was organized in Boston, Mass., on June 21, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of 800 men. Its membership is divided into six classes, viz.:

A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

An Honorary Member shall be a person of acknowledged eminence in some branch of water supply or of engineering.

A Life Member shall be a member whose service to the Association entitles him to special recognition by the Association.

A Junior shall be not less than eighteen years nor more than twenty-five years of age, a student or connected with water supply work.

An Associate shall be either a person, firm or corporation engaged in manufacturing or furnishing materials or supplies for the construction or maintenance of water works.

A Corporate Member shall be either a Water Board, Commission, Company or Municipal Corporation.

The initiation fees and annual dues are as follows:

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Members	$ 3.00
Juniors	1.00
Associates	10.00
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Annual Dues	
Members	$ 6.00
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This Association has seven regular meetings each year, of which five are held at Boston, one in a New England state other than Massachusetts, and one, the annual convention, in a city determined by the Executive Committee.

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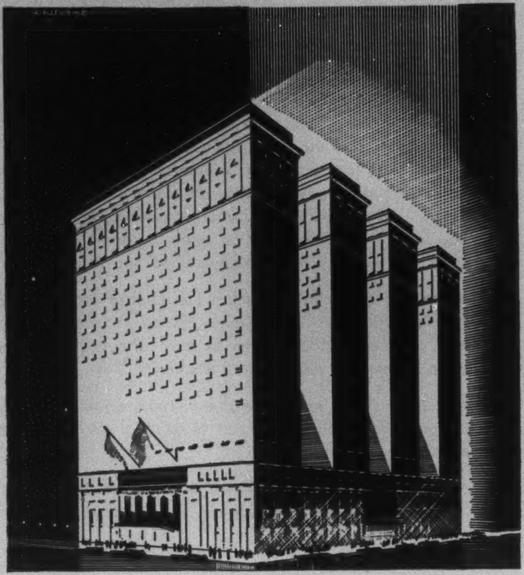

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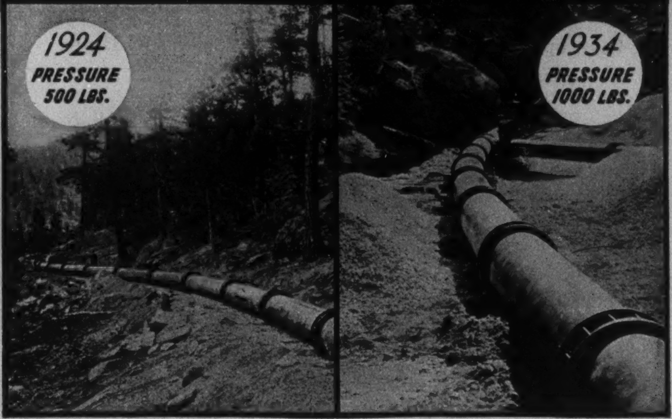

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DRESSER COUPLINGS

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REQUIRED FIRE FLOWS.

BY P. C. CHARNOCK.*

[*Read January 16, 1936.*]

In designing a new water system or rehabilitating an existing system, one of the most important items to be considered is to provide an adequate amount of water for fire supply throughout the various parts of the system. Consideration should be given to the eventual or ultimate requirements of the community even if the original installation is to serve only a portion of the municipality. While the total quantity of water used for fire extinguishment is relatively small, the rate is very high for short periods of time, and systems must be designed to meet this condition.

Rate of Fire Demand. Reasonable fire-protection requirements will vary with structural conditions and with other features affecting the conflagration hazard, and to some extent with the fire department strength of the city under consideration and of its neighbors. Population is naturally not always a fair guide, but on the whole appears to be the best single measure of the approximate quantity of water necessary.

The "Standard Schedule for Grading Cities and Towns of the United States," which was published twenty-one years ago by the National Board of Fire Underwriters, is used throughout the United States and Canada for grading the value of public fire-protection features and is referred to by most consulting engineers as a standard guide in designing water systems; it calls for fire-flow requirements in high-value sections in accordance with the following formula:

$$G = 1020\sqrt{P}\,(1 - .01\sqrt{P})$$

where G = g.p.m. and P = population in thousands. For cities of over 200 000 population, a flow of 12 000 g.p.m. is called for plus 2 000 to 8 000 g.p.m. for a second fire. In residential areas the required fire flow depends on the character of the buildings. Sections where buildings are small and of low height with about ⅓ the lots built upon require not less than 500 g.p.m.; with larger or higher buildings up to 1 000 g.p.m.; and where the district is closely built 1 500 to 3 000 g.p.m. are required and up to 6 000 g.p.m. in densely built sections of 3-story buildings.

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*Manager, Municipal Protection Department, New England Insurance Exchange, Boston, Mass.

In estimating these required fire flows, an allowance is included for probable loss from broken service-connections to buildings burned during the early stages of a bad fire, and such allowance will enable normal pressure conditions to be maintained for a time at least, so that the fire department may not be hampered by lack of pressure as long as there is a chance to control what might develop into a conflagration. During most of the conflagrations in which an estimate of the amount of water used has been possible, the maximum amount of water used in fighting the fire and wasted through broken services and through hydrants left open has been far in excess of what any of these systems may reasonably be expected to deliver while still maintaining normal pressures.

From the foregoing it will be observed that the requirements will vary from a minimum of about 1 000 g.p.m. in mercantile districts in small communities up to 20 000 g.p.m. in very large cities, with a low figure of 500 g.p.m. for residential areas and up to 6 000 g.p.m. in tenement districts.

These flows should be available, even when the consumption on the system is at the maximum daily rate and at a residual pressure of at least 20 lb. per sq. in. in the mains where engine service from the fire department is available. With the rapid advent of automobile pumping engines in fire departments today and in this territory where numerous engines from nearby cities can be mobilized within fairly reasonable lengths of time, there is not the need of attempting to furnish these quantities at 60, 75 or 90 lb. per sq. in. residual pressures as was the case not so many years ago when engines were fewer in number and when horses were used as the motive power. Then, much greater dependence on direct hose lines from hydrants was required.

Calculations can readily be made as to required pipe sizes for new water systems to deliver the amounts called for in various parts of the system, and water works superintendents can easily make tests to determine the amounts available throughout existing systems. There is no necessity for doubt as to the ability of any system to furnish reasonable quantities of water for fire supply. If any of these tests indicate that insufficient amounts are delivered and it is determined that all valves which should be open are in that position and pressures are normal, then the superintendent has concrete evidence to support a plea for appropriations necessary to improve such weak districts.

As a matter of fact, it would be a wise policy for every municipality to conduct or have conducted such tests periodically to obtain information as to the carrying capacity of the water system. Some of the water flowing through our water systems has a very serious tuberculating effect on the piping, thus reducing the carrying capacity greatly, and there is also the ever-present danger that through carelessness valves have been left closed. Tests such as these would give concrete evidence of conditions; determination concerning the valves could then be made, and if the interior pipe conditions are found to be poor, additional mains could be laid or mains replaced or cleaned if this is felt practicable. If any water departments do not have the facilities for making such tests (and these are not expensive), the New England Insurance Exchange is always willing to conduct tests at any time on a request being made.

Duration of Fire Demand. We have been dealing so far with the adequacy of the distribution system as to its ability to deliver fire flow and maximum consumption. The time element over which these flows can be sustained is another phase of the subject of adequacy. For cities of over 2 500 population it is considered that the amounts should be available for a period of 10 hours and below that population for at least 5 hours. Right here the question arises as to whether the supply works and all of the associated pumps, filters, etc. can measure up to this load demand. Elevated storage can easily tend to offset any ordinary deficiencies at the source.

Elevated Storage. There probably is no greater factor in water works practice than elevated storage as a basis for improving fire-protection service of water works systems, and there has been a marked appreciation of the value of such storage in the past few years. This has been brought about in part by the very material changes in design of elevated tanks, a marked lessening in cost, and an appreciation of the saving in operating cost which can be effected through the proper use of elevated storage. Elevated storage is of vital importance to the scheme of water works from the standpoint of fire fighting. A 500 000-gal. elevated storage will provide a 1 500-g.p.m. fire demand for a period of 5 hours. This is equivalent to 6 standard fire streams and is ample for fire conditions in many small communities. These elevated tanks are seldom individually of sufficient capacity to carry over a period of several days duration, which is always a possibility where interruption of supply is due to a broken main, to severe storms interrupting electrical supply, or to break-down of pumps, boilers, etc. But the cost of these large tanks is not so great but that many communities can provide several of them, particularly if the community is in hilly territory where good elevation can be obtained without excessively high framework. Elevated storage is also a vital factor in the design of the arteries of the distribution system. When located on the end of the distribution system opposite the normal source of supply, the delivery capacity at time of fire is greatly increased and sometimes more than doubled for a period of time depending upon the capacity of the tank. It is believed that every water system dependent upon direct pumpage would be materially improved by the addition of elevated storage connecting to the distribution system through pipes of good size.

Present design of elevated tanks permits large structures to be built without great depth to the storage section. This is a very desirable feature as all of the water can be used without serious reduction in pressure. In general, no elevated storage should be allowed to fluctuate more than one-third its capacity. This will assure maintaining good storage for fire service under the most extreme conditions. The elevated tank also permits the installation of covers which is a desirable feature from the health standpoint. One serious objection to tanks and standpipes has been that of nearby residents who have objected to the appearance of the tank. The modern steel tank, while not a thing of beauty, can be made artistic in its general design and can be used as an observation tower at little additional expense.

An essential factor in connection with all elevated storage is the size of the connecting main. This should be big enough to take care of normal fluctuation in consumption demands without material reduction in pressure in the sections of the city containing highest values. For smaller communities, this main should allow for flow at a rate which would permit the delivery of at least two-thirds of the storage in a period of 5 hours. This is based on the assumption of elevated tank storage of less capacity than that necessary to provide fire demand and domestic service for a period of 5 hours. For large reservoirs, connecting mains may be required of a capacity equal to or in excess of the fire-demand rate. Features which must be considered in installing these tanks are the questions of most suitable location, protection against freezing, installation of altitude and check valves, and provision of telemeters or other devices for indicating to the operating force the height of water in the tank.

Maintenance of Supply. Having made provisions for an adequate supply works, pumping plant and distribution system with sufficient hydrants to permit effective use of the supply, attention should be turned to providing for reliably maintaining an adequate fire and consumption demand. Gravity systems with only one feeder into the distribution network are susceptible to hundred percent interruption of service through a break in the main feeder or even by the repair of a valve on this line. In pumping stations, the crippling or necessary overhauling of a pump may so reduce the capacity of the station as to present a serious situation from a fire-protection standpoint. It is not an uncommon occurrence to have two pumps out of service at one time. This applies to high- and low-lift pumping stations or to vacuum pumps and air compressors where used. Power supply whether it be steam, electricity, water power or internal combustion engines is not immune from interruption and thus affects the reliability of supply to the system. Many of our self-contained steam plants, except in large cities, are fast giving way to other forms of motive power, usually electricity. Unless duplicate power circuits on separate pole lines with duplicate transformers are installed or underground service provided, it is almost imperative that internal-combustion-engine-driven pumps be provided in addition.

Filtration Works. In the past 30 years there has been a marked increase in the number of water systems in which the water is filtered. It is probable that within a few years filtering will be considered essential in practically all water systems either to provide a safer water or one which is free of color, odor and certain mineral compounds which are objectionable to the users.

In the early days, filter plants were all provided with by-passes through which water at time of fire could be taken directly from the source of supply, and under these conditions there was little need of having the filter capacity much in excess of the maximum consumption demand nor of providing large storage of filtered water. Today public health demands are such that these by-passes can no longer be installed, and it is the opinion of the underwriting organizations that all water systems which depend upon filtration for the quality of the water should be so designed that the filter system will have sufficient capacity and reliability to assure fire protection at all times without use of a by-pass arrangement. There are many filtration systems which have single pipe lines from low-lift pumps to sedimentation basins and from these basins to the filters and thence to the high-lift pumps. Although pressure conditions are not such as to introduce much likelihood of a rupture of these individual single lines there is always the possibility of breakage of these lines or the interruption of service for a certain length of time when new work has to be done. Our records show cases where pipe lines have been ruptured when driving piles for additions to a pumping station. It is not inconceivable in this day to consider the possible danger which might result from an airplane wreck which might land in or near a filter plant or pumping station. From time to time, material changes must be made in a filter plant such as the replacement of drains, or a general rearrangement brought about by a modification in the method of operation. Settlement of fills, washouts, inadequate foundations of buildings and other conditions may result in broken pipes. It is believed, therefore, that in this modern trend of installing filter plants, particular attention should be paid by the designers and operators to provide as much reliability as possible by duplication of lines or supporting and encasing them in concrete, by segregation of portions of the plants so that total interruption cannot occur, and of providing full adequacy of fire supply either through filter capacity or in storage of clear water at the plant or in elevated tanks and reservoirs.

Other Considerations. Constructing a pumping plant with only a single discharge main or neglecting to provide sufficient valves where duplicate discharge lines are installed are other elements of unreliability. Again, where single arterial feeders are employed, a break in these may jeopardize the protection of the entire plant due to the fact that the remaining minor feeders are incapable of carrying the load.

Many of these elements of unreliability can be overcome in whole or in part by constructing adequate storage on the system. By adequate storage it is felt, so far as breaks in steam or oil lines or electric supplies are concerned, that if the storage is such as to provide, in conjunction with any remaining operating pumping units, 10 hours of fire flow at the end of two days of maximum consumption a safe standard is met. For the break-down of pumps or breaks in pipes 10 hours of fire flow at the end of five days of maximum consumption demand should be cared for by storage.

A water system which fulfills these requirements of adequacy and reliability should permit a water works superintendent to take considerable pride in its accomplishments, and with a competent operating force should give him some peace of mind when he returns to his fireside at night.

AUTOMATIC SPRINKLER SYSTEMS AND THEIR WATER SUPPLIES.

BY HENRY E. HALPIN.*

[*Read January 16, 1936.*]

The first automatic sprinkler to be used extensively in practice was invented in 1874. Most of the early developments in automatic sprinkler systems took place in New England,the installations being made in textile mills. Even now after sixty years the public in general is still ignorant of the wonderful record, value, and functions of automatic sprinklers.

It is the purpose of this paper to describe these systems and the principal types of apparatus used in them so that the water authorities may have a better understanding of the equipment that is connected to their mains. Time permits mention of only the most important devices. Some discussion will be given to water supplies and friction losses in sprinkler piping. The paper will also discuss the flow tests made by the Inspection Department of the Factory Mutual Fire Insurance Companies and a few types of special assistance given to industries in the maintenance of automatic sprinkler equipments.

FUNCTIONS OF AN AUTOMATIC SPRINKLER SYSTEM.

Sprinklers discover and control incipient fires. Every sprinkler head is a thermostat, plus an extinguisher. An automatic sprinkler system provides an army of mechanical watchmen always on duty throughout a property. They operate promptly at the start of the fire, discharge water directly onto the seat of the fire regardless of heat, smoke and gas which are serious handicaps to firemen. Because of their method of operation, they use water most effectively and cause a minimum of water damage.

Records of the National Fire Protection Association show the following experience in 69 000 fires in sprinklered properties:

80 per cent. of fires extinguished or held in check by eight sprinklers or less.

85 per cent. of fires extinguished or held in check by twelve sprinklers or less (about the equivalent of one hose stream).

90 per cent. of fires extinguished or held in check by twenty sprinklers or less.

95 per cent. of fires extinguished or held in check by fifty sprinklers or less.

Sprinklers minimize the spread of fire and of damage to adjoining property. They also act as barriers to the spread of fires originating elsewhere. Automatic sprinklers prevent interruption of business and preserve taxable property. They protect human life in factories, schools, hospitals, jails and theatres.

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*Engineer, Associated Factory Mutual Fire Insurance Companies, Boston, Mass.

AUTOMATIC SPRINKLER HEADS.

An automatic sprinkler may be defined as a device, that, when heated to a predetermined point, automatically releases and distributes a spray of water about 8 ft. in diameter. The essential parts of an automatic sprinkler are a nozzle, a releasing device and a deflector to distribute water on the ceiling and contents. In all modern sprinkler heads, with two exceptions, parts are released by the melting of solder. Over 700 different kinds of sprinklers have been made; 21 are now approved by the Factory Mutual Fire Insurance Companies.

Grinnell Glass Button Sprinkler. There have been more of the Grinnell sprinkler, glass button pattern, installed than of any other make. This head was invented in 1890 and improved in 1903 from which date it was installed continuously until 1935 by the Grinnell Company (Fig. 1).

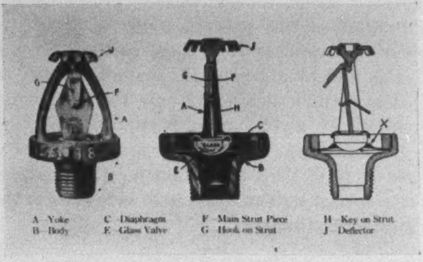


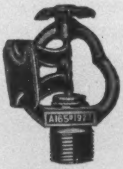
FIG. 1.—GRINNELL GLASS BUTTON SPRINKLER.

The frame is cast bronze. The orifice is a ½-in. hole in a phosphor bronze diaphragm kept closed by a semi-spherical glass disc held in place by a fusible link. The orifice is faced with a soft solder to obtain a tight joint. The releasing device is a three-piece soldered link, consisting of a strut, a hook piece and a key piece. The deflector is rigidly attached to the top of the sprinkler frame and has a row of teeth projecting from its circumference to aid in distribution.

Tests for Approval of Sprinklers. Laboratory tests for approval require that the sprinkler:

(1) Must not leak even at 500 lb. per sq. in. water pressure.
(2) Soldered joint must be strong enough not to give way prematurely.
(3) Must give uniform distribution of water on both ceiling and floor.
(4) Must withstand reasonable abuse.
(5) Must open within 5°F. of temperature rating.

Lever and Link Type Sprinkler. The lever and link type of releasing device is most commonly used in the present approved sprinklers. The Hodgman Type A sprinkler, made by the Hodgman Mfg. Co., Taunton, Mass., is typical of the lever and link type (Fig. 2). The frame, as in all approved sprinklers, is bronze which resists corrosive atmospheres fairly well and has suitable tensile qualities. The nozzle is a ½-in. ring nozzle, this type of waterway being used in all present approved sprinklers with one exception. The soldered link is composed of two rectangular bronze plates with rectangular key fitting a groove in each leaf and extending the entire width. Twenty pounds tension is set up in the link. The valve consists of a flat stamped bronze cap and a copper disc. The deflector is stamped bronze with eight square tines and four holes.



HODGMAN A
1921—Approved
FIG. 2.

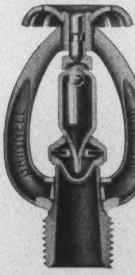


FIG. 3.— GRINNELL QUARTZOID BULB TYPE SPRINKLER — ISSUE C.

Bulb Type Sprinkler. This sprinkler is of particular interest because the releasing device is a bulb composed largely of quartz, rather than the usual soldered link (Fig. 3). The bulb is hermetically sealed at each end and is filled with a liquid of the carbon tetrachloride group except for one small bubble of air. When the sprinkler is heated the liquid expands until it bursts the bulb which releases the valve. Different operating temperatures are obtained by varying the size of the bubble. The valve cap assembly comprises two bronze parts which form a closed chamber and in which are located two flat spring-steel discs. These provide a cushioned mounting for the bulb and prevent abnormal stress in the bulb due to slight manufacturing inaccuracies. The deflector has fourteen prongs evenly spaced.

Temperature Ratings of Sprinklers. For room temperatures up to 100° F., solder-type sprinklers with a temperature rating of 155–165° F. are used. This affords a margin of safety of at least 50° F. Sprinklers for driers, skylights, locations near steam coils, and other warm occupancies are available with the same margin of safety as indicated in the table below. To facilitate identification of high-temperature sprinklers, the frames of these heads are painted different colors as noted in the table. The margin between maximum safe room temperature and operating temperature is less for bulb type than solder type sprinklers, because bulb type sprinklers always operate at the rated temperature while solder is subject to fatigue or "cold flow" when exposed to temperatures slightly above the safe room temperature.

TEMPERATURE RATINGS OF SPRINKLERS, AND SAFE ROOM TEMPERATURES.

Designation of Sprinklers.	Color of Frame.	Temperature Rating — ° F.		Safe Room Temperature — ° F.
		Solder Type.	Bulb Type.	
Ordinary	Bronze	155–165	135	100
Intermediate	White	212	175	150
Hard	Blue	286	250	225
Extra Hard	Red	360	325	300

Volume Discharged by Sprinklers. A rule of thumb which has been widely used is that the average discharge from a group of sprinklers is 20 g.p.m. per head. Where the pressure at the sprinkler is known, or can be estimated, the volume of water discharged can be more closely estimated from the table below. The discharge from sprinklers of different makes varies somewhat, but the variation is slight, and the figures in the table represent a fair average.

VOLUME OF WATER DISCHARGED BY SPRINKLERS AT VARIOUS PRESSURES.

Pressure at Sprinkler, lb. per sq. in.	Discharge Average Value, g.p.m.	Pressure at Sprinkler, lb. per sq. in.	Discharge Average Value, g.p.m.
5	12	50	41
10	18	60	45
15	22	70	48
20	25	80	52
30	31	90	55
40	36	100	58

For comparison, one ordinary 1⅛ in. fire stream discharges about 250 g.p.m. at a nozzle pressure of 45 lb. per sq. in.

AUTOMATIC SPRINKLER PIPING SYSTEMS.

Typical Arrangements of Piping and Heads. Figure 4 shows automatic sprinklers installed under plank on timber construction. The feed main is located at the center of the building and one branch line is located in the center of each bay across the building. Sprinklers in this type of construction are spaced to give a maximum of 100 sq. ft. per head, but heads are never over 12 ft. apart in the bay. Under reinforced concrete, sprinklers can be 10 ft. apart in each direction, a maximum of 100 sq. ft. per head.

Joisted construction is more difficult to protect because the sprinkler discharge to the ceiling is broken up by the joists. Branch lines are located across the joists with sprinklers not over 8 ft. apart. Branch lines are not over 10 ft. apart so that one sprinkler is laid out for a maximum of 80 sq. ft. Sprinklers are staggered, that is, the heads on any line are located opposite points half way between heads on the adjacent lines. The purpose of staggering is to permit the discharge of water into a greater number of pockets between joists.

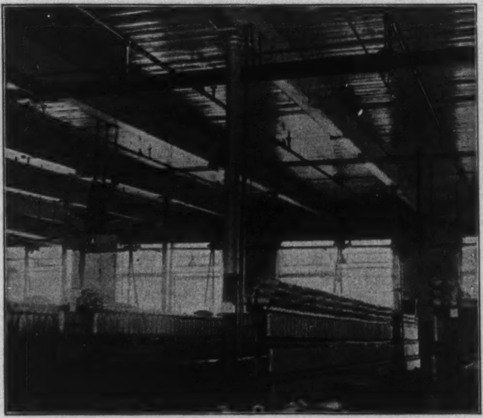


FIG. 4.—SPRINKLERS UNDER PLANK ON TIMBER CONSTRUCTION.

Sprinklers should be upright above pipes. Sprinklers with deflectors 4 to 8 in. below smooth plank or concrete ceilings, or joists under joisted construction, will give the best distribution with pressures likely to obtain during a fire. Deflectors should under ordinary conditions be located within these limits. Special construction may make modifications necessary, but in no case should deflectors be less than 3 inches or more than 10 inches below joists or ceiling.

Sprinkler Control Valves. Sprinkler control valves should be located so that they can be reached in an emergency. Wherever there is yard space, it is customary to control sprinkler equipments by indicator-post gate valves located outdoors and preferably 40 ft. from buildings. Where conditions are such that indicator posts cannot be used, as when a driveway occupies the whole available space, an outside screw and yoke gate valve can be installed in a pit. At city properties, where no yard room is available, accessibility of controlling gates may be secured by installing them in a well cut off stair tower or non-combustible valve room entered from the outside. Floor valves and control valves inside of buildings on sprinkler risers should be avoided especially where the construction is combustible. In non-combustible buildings, say more than seven stories high, it may be advantageous to control sprinklers in each floor by a valve located in a stair tower. Occasionally in city properties, reasonable safety of control may be obtained by locating the sprinkler valve on the opposite side of a fire wall from the sprinklers controlled.

When none of these methods of control can be used, it may be necessary to depend on the water works gate in the street for outside control. In these cases, close coöperation between the fire and water departments is necessary. In any event the response of water department representatives to two-alarm fires is advisable.

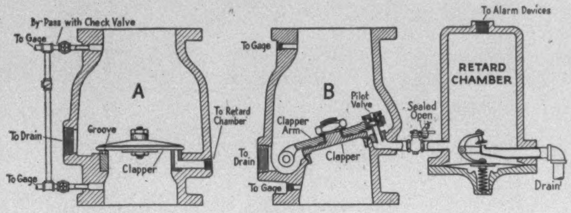


Fig. 5.— Typical Alarm Valves and Retard Chamber.

Alarm Valves. Water-flow alarms are valuable additions to the fire-protective equipment. Though not necessary on every sprinkler riser, they are strongly recommended for buildings which have contents susceptible to fire or sprinkler leakage. They are a desirable supplement to watchman's service because, when there is no alarm, the interval between rounds may under some conditions allow considerable time to elapse before a fire or opened sprinkler is discovered.

Alarm check valves are similar in design to regular check valves except that they have either a small auxiliary pilot valve or a divided seat ring which admits water to the alarm device, through a retarding chamber, when the flow equals or exceeds that for a single sprinkler (Fig. 5). The retarding chamber is designed to prevent false alarms by flows of short duration, by delaying the alarm at least 10 seconds and not over 30 seconds. A circuit closer or water-motor alarm or both may be used for alarms.

In Figure 5, "A" is the divided seat ring type. Flow for one or more sprinklers raises the clapper, and water enters the groove in the seat ring (normally covered by the clapper) and passes to the retard chamber and alarm devices. Small flows and surges raise the main clapper momentarily or pass through the bypass if installed. "B" is the pilot-valve type of alarm check valve. Flow for one or more sprinklers raises the clapper and water enters the pilot orifice and passes to the retard chamber and alarm devices. Small flows and surges raise the main clapper slightly but not enough to uncover the pilot orifice. Part of the water entering the retard chamber from the alarm valve discharges through the drain; the remainder gradually fills the chamber until the weight of water depresses the diaphragm, closing the drain valve, thereby causing the water pressure to operate the electric alarm or the flow to operate the water-motor alarm.

The American District Telegraph Company and its subsidiary companies maintain a central-station supervisory service in most of the principal cities in the United States. Sprinkler alarms, water levels in gravity tanks, air pressures in dry pipe systems, watchmen's stations, fire alarm boxes, burglary alarms and other types of supervisory service are provided. When an alarm is received at the central A.D.T. station indicating flow of water, the public fire department is immediately summoned.

This company is now installing its Type 497 Differential Diaphragm Water-Flow Detector rather than an alarm check valve similar to that previously described. This device includes a small pressure tank, a slow-leak element and a diaphragm and is connected to the sprinkler system through a ½-in. pipe connection above a weighted or differential check valve in the sprinkler riser. It is actuated by a sudden drop in the sprinkler system pressure when a sprinkler head opens, acting on one side of the diaphragm only. In case of a sudden increase in system pressure, the diaphragm is deflected toward the pressure tank side and no signal is given. By retarding the alarm one or two seconds, false alarms due to water hammer and surges can be practically eliminated.

SPRINKLER SYSTEMS FOR UNHEATED BUILDINGS.

When a building is not sufficiently heated to prevent freezing of wet-pipe sprinklers, a dry-pipe system is installed. The sprinkler pipes are maintained under air pressure, and a dry-pipe valve is installed in the sprinkler riser, generally at the lowest level. These systems are less desirable than wet-pipe systems, because water is not available at the sprinkler heads as promptly, the systems are more complicated, and more maintenance is necessary.

A dry-pipe valve is a device for holding back the water supply to a sprinkler system until a sprinkler operates and then opening automatically and admitting water. Air pressure in the sprinkler pipes above the dry-pipe valve hold it normally closed.

There are two types of dry-pipe valves: the differential and the mechanical valve. The differential valve is essentially a check valve with two seats, the area above the upper, or air, seat being much greater than that below the lower, or water seat. This difference in areas creates a pressure differential of about 6 to 1 so that 15 lb. per sq. in. of air will hold back about 90 lb. per sq. in. of water pressure. Air pressure normally carried in differential dry-pipe valves should be one-sixth the water pressure plus 10 to 15 lb. per sq. in. Most of the present approved dry-pipe valves are of the differential type.

The differential dry-pipe valve shown in Figure 6 has but two moving parts, a pivoted counterweighted clapper and a latch to hold it in the open position. The air and water clappers are combined in a single bronze disc. The clapper arm is counterbalanced to assist in opening.

All dry-pipe valves have an intermediate chamber or neutral space between the air and water seats to which an alarm is connected. Any dry-pipe valve may have an electric or hydraulic alarm or both. An automatic drip valve in the intermediate chamber takes care of any small leakage by the water seat.

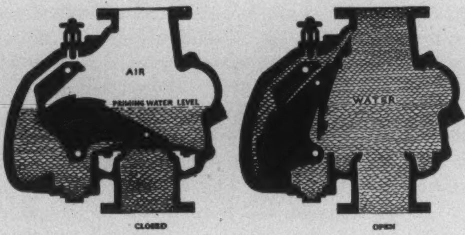


FIG. 6.— SIMPLIFIED SKETCH OF DIFFERENTIAL DRY PIPE VALVE.

Differential dry-pipe valves have a latch to hold the clapper off its seat after tripping and prevent reseating. It may be possible to water-column a dry-pipe valve. For instance, in a differential valve with a differential of six to one, a water pressure of 10 lb. per sq. in. (23 ft.) on top of a seated valve would column it against 60 lb. per sq. in. of water supply pressure. In actual practice little or no trouble is experienced from water columning.

In the mechanical type of valve, the water clapper is held on its seat by air pressure exerted on the air clapper acting through a series of levers. The dry-pipe valve is designed to open at a predetermined air pressure. Air pressure of 25 to 35 lb. per sq. in. is carried regardless of water pressure.

An accelerating device, known as an "accelerator" or "exhauster" can be attached to a dry-pipe valve and cause it to trip within a few seconds after a sprinkler opens, thus making a dry-pipe system approach a wet system in efficiency. Because of this improved efficiency, the size of a system equipped with an accelerator or exhauster can be increased from the usual maximum of 500 to 750 gal. air capacity.

WATER SUPPLIES FOR SPRINKLER SYSTEMS.

Pressure and Volume. It is essential for good protection that sprinklers be so arranged under combustible construction that the ceilings will be protected, that is, pretty thoroughly wetted by water from the sprinklers. A sprinkler, with the deflector 4 to 6 in. below a smooth flat ceiling and with a water pressure of 10 to 25 lb. per sq. in. will wet a ceiling area from 6 to 8 ft. in diameter. With higher water pressures, more water will be discharged, but the thoroughly wetted area will not be greatly increased. With 5 lb. per sq. in. at the head, a smooth ceiling area of about 4½ ft. diameter will be wetted.

A flowing pressure of 5 lb. per sq. in. *at the sprinkler* is the minimum which could be considered for reasonably good distribution. For thoroughly good distribution, 10 lb. per sq. in. at the sprinkler are necessary. Effective protection from sprinklers, however well arranged they may be, cannot be expected unless they are supplied with water at fair pressure and ample volume. A primary supply which cannot be depended on to supply at least 30 sprinklers without dropping the pressure at the most remote head below 5 lb. per sq. in. cannot be considered adequate for even a small plant. This draft may be considered as not less than 500 g.p.m. One thousand gallons or more per minute for the same pressure conditions may be needed for larger properties or where exposure is serious. Additional volume is needed where the public water must also supply hose streams.

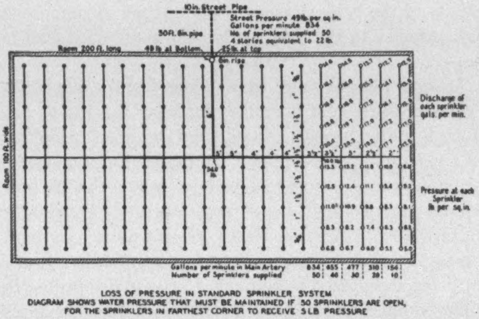

10in. Street Pipe
Street Pressure 49lb. per sq in.
Gallons per minute 834
No. of sprinklers supplied 50
4 stories equivalent to 22 lb.
50 ft. 6in. pipe
Room 200 ft. long
49 lb. at Bottom.
25 lb. at top
6in. rise
Room 100 ft. wide
34.0 lb.
5" 5" 4" 4" 4" 2½" 2½" 3" 2½" 2"
Discharge of each sprinkler gals. per min.
Pressure at each Sprinkler lb per sq. in.
Gallons per minute in Main Artery 834 655 477 310 154
Number of Sprinklers supplied 50 40 30 20 10
LOSS OF PRESSURE IN STANDARD SPRINKLER SYSTEM
DIAGRAM SHOWS WATER PRESSURE THAT MUST BE MAINTAINED IF 50 SPRINKLERS ARE OPEN, FOR THE SPRINKLERS IN FARTHEST CORNER TO RECEIVE 5 LB PRESSURE

FIG. 7.— LOSS OF PRESSURE IN STANDARD SPRINKLER SYSTEM.

Supplies from two independent sources are necessary for thorough protection of large and valuable properties. The water supply that sprinklers have first, is referred to as the "primary" supply and the other as the "secondary" supply. If one supply is out of service for repair or because of an emergency interruption, the other supply will be available. Another reason for dual supply is that it gives additional water to supplement a primary supply of moderate strength in case of a large fire.

Figure 7 shows a modern sprinkler layout of 200 heads in one area and indicates that a calculated pressure of 49 lb. per sq. in. is necessary in the street main to maintain a pressure of 5 lb. per sq. in. at the most remote sprinkler when 50 sprinklers are operating in the fourth story. The friction loss in a branch line of five heads averages 6 lb. per sq. in. The pressure at the top of the riser is 25 lb. per sq. in. Loss of pressure due to elevation is 22 lb. per sq. in. The quantity flowing is 834 g.p.m., an average of 17 g.p.m. per sprinkler.

If fire department pumpers are drawing from public hydrants for hose streams, the street-main pressure will be reduced further. Where the water supply is limited, pumpers may rob the sprinklers of adequate pressure. On the other hand, fire department pumpers can supplement the water supply to sprinklers by pumping into the siamese or fire department connections with which most sprinkler equipments located in city properties are provided. In many of the large cities it is the practice to connect one hose line from the first pumper to the fire department connection and pump water, if necessary.

Public Mains. One or more connections from a reliable public water system of good pressure and adequate capacity furnish an ideal primary supply. A high static pressure should not be a criterion by which the efficiency of the supply is determined. A system with a static pressure of say 80 or 100 lb. per sq. in., but which will not deliver the fire flow previously discussed cannot be considered a satisfactory primary supply for sprinklers. The great improvements in public water systems during the past 15 or more years including supplies, storage, pumping equipment and distribution systems, are being recognized in layouts of private fire-protection by specifying secondary supplies of smaller capacities and by accepting single water supplies for larger properties than formerly.

Fire Pumps. A well located fire pump is, under most conditions, the most satisfactory source of the secondary supply, as with ample suction supply it is capable of maintaining a high pressure over a long period of time. Steam, rotary and centrifugal fire pumps of approved design are available. The majority of new pumps are of the centrifugal type driven by an electric motor. Few new rotary pumps are being installed as they are low-speed pumps for water-wheel drive. Steam-driven pumps have been replaced with motor-driven units at many plants where steam power plants and high-pressure boilers have been discontinued. Gasoline-engine drives have been used to a limited extent where other reliable sources of power were not available or as auxiliary units in connection with other pumps.

The most common capacities of fire pumps are 750 and 1000 g.p.m. Suction supply for pumps should be large enough to cope with any fire that might reasonably be expected. A two-hour suction supply should be considered a minimum, even at the smallest plants. At many plants a suction supply sufficient to operate the pump at rated capacity for three hours should suffice. At the largest industrial properties an inexhaustible suction supply is desirable for good protection. Water supplies should be arranged to conform to the regulations or standards of the State Department of Health.

Gravity Tanks. Where a good city water system is not available for a primary supply or where pumps are not feasible for the secondary supply, elevated tanks of steel or wood are used. The conditions at each plant must determine the size and height of the tank. The most common sizes of sprinkler tanks are 25 000, 50 000, 75 000 and 100 000 gal. The 25 000-gal. tank is used mainly for the protection of small city properties consisting of a single building. This size tank should be reserved for sprinklers only and not connected to hydrants or 2½-in. hose standpipes. The 50 000-gal. tank is the most common size installed at the smaller industrial properties and the 100 000-gal. tank at the largest properties.

The limitations of sprinkler tanks are (1) the fixed moderate capacity as compared with public mains, and (2) the moderate pressures particularly for hose streams as compared with fire pumps. In any case the bottom of a sprinkler tank should be not less than 25 ft. above the highest sprinklers. When tanks supply hydrants, the bottom should be not less than 75 ft. and preferably 100 ft. above yard level.

Pressure Tanks. Pressure tanks are sometimes used to give adequate pressure in upper stories when public water pressure is low, or when the other supply is taken from a small-capacity gravity tank. Pressure tanks are generally located in a heated enclosure on the roof and filled two-thirds full of water and one-third full of air at 75 lb. per sq. in. pressure. They are necessarily limited to comparatively small sizes, the most common of which are as follows:

4 500 gal.	6 000 gal.	7 500 gal.	9 000 gal.
(3 000 gal. water)	(4 000 gal. water)	(5 000 gal. water)	(6 000 gal. water)

Two or four pressure tanks in combination with gravity tanks and fire department connections are installed for the protection of many reinforced-concrete loft buildings in New York City. Complete central-station supervisory service is provided for sprinkler valves, water flow, high- and low-water levels in tanks, air pressures, and tank-water temperatures at these properties. Hose protection is by standpipe with separate water supplies.

CARE AND MAINTENANCE OF SPRINKLER SYSTEMS.

Inspection of Sprinkler Control Valves. At all properties insured in the Factory Mutual Companies, a weekly inspection of all fire-protective equipment with detailed records is made by a responsible employee. The equipment is inspected every three or four months by a representative of the Inspection Department. In addition, occasional inspections of special equipment are made by Laboratory engineers.

The most important single item in the inspection is the sprinkler control valve, because sprinklers are useless without water. A system of joint-sealing of sprinkler control valves was inaugurated many years ago as a means of reducing the number of valves found closed by inspectors. A light sealing wire is attached to the valve by the property owner and the sealing button is snapped into place by the inspector. This system has worked remarkably well and reduced the number of closed valves to a low figure. Notification of the insurance company whenever a sprinkler valve is shut off for repairs or emergency also permits a closer check on restoration of protection and helps to avoid a shut valve.

A flow test through the riser drain pipe will indicate that the sprinkler system is under pressure and will detect a shut gate, frozen or obstructed pipe. Such tests are made occasionally by the property owner and at each riser during the visit of the insurance company inspector. Alarms are tested weekly.

The water works gates on connections from street mains to sprinkler equipments are important as they control the main supply. These valves, along with other gates, are being checked annually in numerous public water systems. This is certainly a desirable practice.

Special Investigations. Some of the special assistance given by the Inspection Department to aid in excellent maintenance of sprinkler equipments will be mentioned very briefly. Slightly over 5 000 sprinklers were examined or tested for reliability in the sprinkler laboratory in 1935. Inspectors are constantly alert to observe sprinklers which are subjected to unfavorable conditions such as overheating, corrosion and deposits of foreign material. The number of sprinklers which require checking is remarkably small, considering the millions in service.

Tripping tests of all mechanical-type dry-pipe valves (about 2 000) were witnessed by inspectors in 1935.

Annual Water Flow Tests. An annual capacity fire-flow test is made between April first and October first at each insured property. Each fire pump is tested to capacity through hose streams, and a flow is drawn from each sprinkler tank. Connections to public water supplies are tested separately to determine that each supply is open, and a final draft from all city connections together is made to observe the total volume available at effective fire pressure and to compare with past performances.

In concluding, I wish to take this opportunity to thank the water works superintendents for their coöperation with our representatives in making tests at public hydrants. These tests are very important when planning water supplies for a proposed sprinkler installation or when a question has arisen regarding the volume of water available in the street.

CUSHNOC HEIGHTS HIGH SERVICE, AUGUSTA WATER DISTRICT.

BY S. S. ANTHONY.*

[Read September 19, 1935.]

A general description of the system of the Augusta Water District is desirable in order to give an idea of the necessity of the new high service which has been installed during the past summer.

The city of Augusta is situated along the banks of the Kennebec River about 30 miles from its mouth. The principal business section abuts on Water Street which lies directly on the bank of the river at El. 40 and extends for about one half mile below the Falls on the west bank. Joining the Kennebec River at the north end of Water Street is a small stream known as Bond Brook, which sharply divides the west side of the city. To the South of Bond Brook lie the business section, the Capitol and the principal residential section, with the Air Port plateau bounding the residential section on the West at El. 350.

The peninsula between the Kennebec and Bond Brook contains the principal industry, a cotton mill, and the dwellings of most of the mill hands, as well as practically all of the French-speaking population and the largest rural ward of the city. This peninsula is thickly populated and rises to El. 368, with dwellings as high as El. 300 in less than three quarters of a mile from Bond Brook.

The water supply of Augusta is obtained from Carleton Pond, a body of water of about 200 acres situated nine miles west of Augusta at El. 333 in Readfield and Winthrop.

Water flows from the pond a distance of about two miles to a booster station in East Winthrop at El. 174, thence through a 24-in. cast-iron main to two distributing reservoirs adjacent to the Air Port at El. 324, providing a gravity supply of approximately 1¼ m.g.d. The deficit in rate is made up by operating the booster station during such hours as necessary to provide an average daily supply rate of about 2 m.g.d. The two distributing reservoirs have a total capacity of 16 m.g.; one contains 10 m.g. and the other 6 m.g.

From the distributing reservoirs a 12-in. cast-iron pipe leads to the center of the business section and thence across the river on the highway bridge in two 10-in. pipes to the east side grid. A 16-in. cast-iron pipe leads directly to the cotton mill, a relic of the days previous to 1903 when the supply for the city was taken from the Kennebec River by the pumping station in the mill yard, using this 16-in. pipe as a force main. A third supply is taken from the pumping station main at a point about one mile west of the reservoirs, thence across country to the Winthrop Road, through the residential section to the Capitol.

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*Engineer and Superintendent, Augusta Water District, Augusta, Me.

The peninsula previously mentioned between the brook and the river is called from old maps, Cushnoc Heights. This is the section with which we are concerned in this paper.

From the time of the first construction of a water supply in Augusta, this section of about one half square mile, containing over 500 dwelling houses, was supplied by means of a single 6-in. line starting at the cotton mill and running up Northern Avenue to Franklin Street, with a circuit of 6-in. pipe in Washington and Franklin Streets back to Northern Avenue about 1000 ft. from the cottom mill. There were nine hydrants on the circuit. Intermediate and cross streets were all supplied with 2-in. pipe.

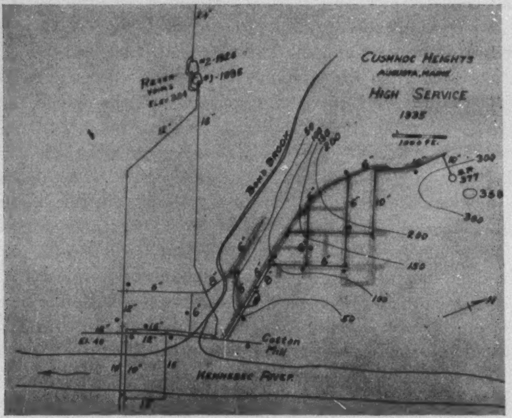

Fig. 1. — Cushnoc Heights Area.
Augusta, Maine.

In 1926, an 8-in. line was laid from the cotton mill to the junction of Northern Avenue and Washington streets. In 1932 and 1933, 6-in. pipes were laid in Jefferson Street from Washington Street to Northern Avenue and in Oxford Street from Northern Avenue to Franklin Street providing a better grid system and a slightly better supply with the addition of the 8-in. pipe in Northern Avenue.

We still were unable to supply adequate fire protection, however. A single fire stream from any of the lower hydrants robbed both services and hydrants on the higher levels of water and at times created a vacuum, with consequent collapse of range boilers. The fire flow from the three higher hydrants was negligible. As time went on, the section became more closely built up, with many multifamily tenement houses and increasing demands for water. A situation developed which made possible a conflagration of at least as large proportions as the fire at Auburn, Me., in 1933, with absolutely no chance to obtain water from natural sources due to great elevation. The need for additional storage and pressure had been recognized for some years and discussed more or less seriously among the Trustees of the District for a year or two, with determination to remedy the situation when possible.

Last fall, an application was made to the local ERA administrator for a survey party to make surveys for possible sites for a standpipe in this section as well as in five other sections where additional storage will be necessary in the future. No action on our application being taken for some months, we engaged our own survey party, recruiting unemployed highway engineers, and completed the surveys late in the winter. The design for the Cushnoc Heights system was then commenced.

In February, the State Planning Board requested a list of possible projects which might be undertaken if and when funds were made available by the Federal Government or other agencies. These six standpipes were listed at that time. The situation in the PWA being very unsettled and no definite plan of financing or even application being forthcoming, the author was instructed by the Board of Trustees at its May meeting, to proceed with the design and construction of the Cushnoc Heights project.

It was decided that two days supply with an increase of about 30 lb. per sq. in. in static pressure was the object to be attained. A tank of 300 000 gal. capacity with the water line at El. 377 seemed to fill the bill, together with additional mains of larger capacity and a booster station located near the large supply at the mill yard. (Figs. 2 and 3.)

The booster station was constructed of brick on a concrete foundation, with the sill at El. 60.0. Since it is located on a main highway to Waterville it seemed desirable to make it as attractive as possible. Accordingly, the design was turned over to Bunker & Savage, a local firm of architects, who provided us with plans and specifications for a small brick Colonial building with cream trim and slate roof, with a floor area of 12 by 20 ft. inside and a ceiling height of 7½ ft. This building was constructed by Stewart & Williams at a cost of $2,200. A white picket fence was provided around the whole lot, in a style in keeping with the design of the building, making an attractive building which not only will not depreciate values in the neighborhood, but will tend to enhance them.

The intake pipe was connected with both the 6- and the 8-in. mains in front of the station, brought together into a 10-in. pipe leading into the building. The 10-in. pipe was there divided into two 6-in. pipes, one for each pump. The service pump is a Cameron motor pump, furnished by Ingersoll Rand Co. with a capacity of 250 g.p.m. at 73-ft. head, controlled by a General Electric pressure control mounted at the standpipe giving

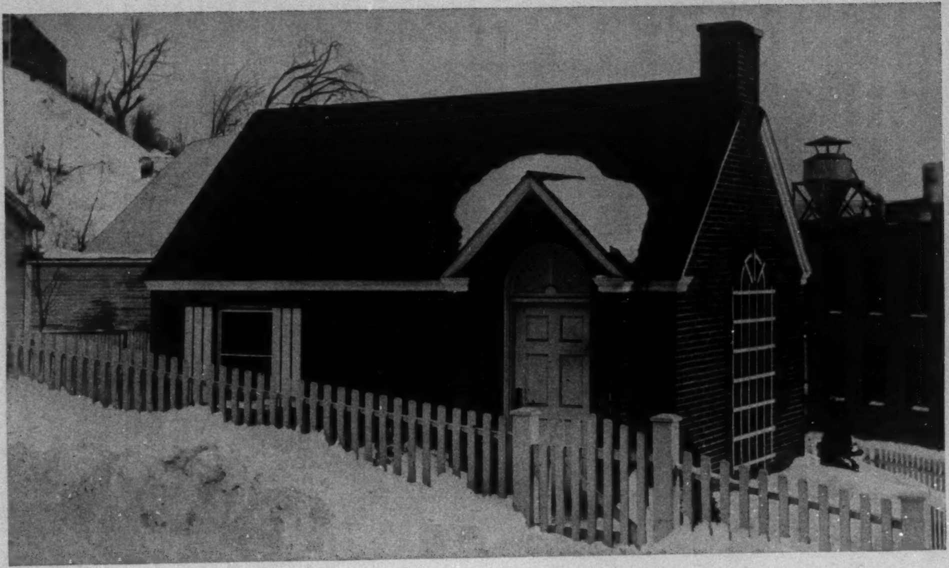

Fig. 2. — Cushnoc Heights Booster Station.

a fluctuation of 12-in. in water level. The efficiency of this pump is 86 per cent. The fire pump is a Cameron centrifugal pump, operated by a 40-h.p. General Electric motor on one end and an H-217 Buda gasoline engine on the other. This pump is manually controlled and has a capacity of 750 g.p.m. at 129-ft. head. The difference between the pumping heads of the two pumps takes into account the friction in the rather old distribution mains between the pumps and the standpipe.

The electrical controls were furnished by the General Electric Company, all mounted on a steel switchboard furnished by them. This board also includes the recording instruments of a 10-in. by 4 in. Venturi Chronoflow meter of the Builders Iron Foundry, and a Foxboro recording pressure gauge. All recording instruments are operated by Telechron electric clock mechanisms.

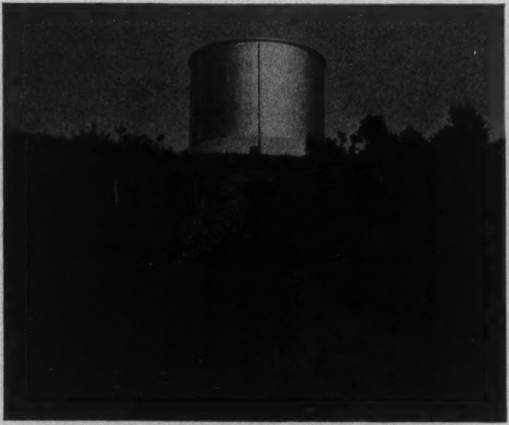

Fig. 3. — Cushnoc Heights Standpipe.
Capacity 300 000 gals.

Both pumps are equipped with Chapman non-slam type check valves, and a 3-in. Ross pilot-operated relief valve is set in the discharge line. The discharge from both pumps is brought together into a 10-in. line leading through the Venturi meter back into the 6- and 8-in. lines in Northern Avenue, with a gate valve set in these lines between the intake and discharge connections.

The standpipe foundation is constructed as a circular retaining wall set on solid ledge with an inside diameter of 43 ft., or one foot larger than the tank itself. The wall is 24 in. thick on top and batters 1 in. in 12 on the outside. Under the tank, and forming part of the foundation, is a valve chamber 6 ft. square and 8 ft. high, penetrating 5½ ft. into solid ledge. This ring or retaining wall was filled with broken rock from the trench excavation to within 1 ft. of the bottom of the tank. After this broken rock was placed by hand as compactly as possible, over forty loads of screened sand were dumped onto the rock surface and washed into the voids by means of a high-pressure hose with a ½-in. nozzle. The top 12 inches between the rock and the tank were then filled with bank-run gravel, well dampened and compacted by running a loaded truck around and around and back and forth. After thorough compacting, the gravel fill was trimmed to grade by hand and the bottom plates of the tank were placed and welded.

The tank is 42 ft. in diameter and 30 ft. high, with a capacity of 300 000 gal. The bottom and bottom course are 5/16 in. thick and the four upper courses are of ¼-in. sheets. The top ring or stiffener is a 10 by 3 by 5/16-in. Z bar. All seams were welded electrically. The seams in the bottom were welded on the inside; the circumferential seams welded on the outside and spot welded on the inside; the vertical seams welded on both sides; and the joint between the bottom and sides welded on both sides. A 20-in. manhole was set in the south side.

The water supply comes in and out through a 10-in. cast-iron pipe with a lead joint between the pipe and the bottom plate of the tank. This pipe is also used as an outlet pipe with the top of the bell mouth set 15 in. above the floor. Adjacent to the inlet and outlet pipe is a 6-in. pipe set flush with the floor and used as a drain. It is led down the hill to Northern Avenue and discharges into the gutter there. A 1-in. brass pipe runs from 6 in. above the floor into the valve chamber where is located the control switch which operates the service pump. Leading to the tank are 2 000 ft. of 10-in. pipe, connected at two points with the distribution system through Y connections and long-turn fittings.

The cost of the complete project is about $18 000, divided as follows:

Standpipe, erected and painted	$3 850.00
Standpipe foundation	2 200.00
Booster station	2 400.00
2 000 ft. of trench, 26 in. wide and 6 ft. deep, including about 150 cu. yd. of ledge	3 300.00
Fire pump. Complete on common bed plate, with gasoline engine and electric motor	1 163.00
Service pump	228.00
Control panel or switch board complete with all control devices	1 000.00
2 000 ft. of 10-in. pipe laid by our crew	3 000.00
Electrical installation	235.00
Chronolo meter	490.00
Miscellaneous labor	134.00
Total cost	$18 000.00

The contract for the construction of the station, standpipe foundation and trenching was awarded to Stewart & Williams, Inc. of Augusta on May 24; work was started on May 25; and their contract was completed about August 1. The Pittsburgh Des Moines Steel Company furnished and erected the standpipe, beginning on July 10 and completing it ready for filling and testing on August 19. Due to delay in the delivery of the considerable number of special fittings required, the pump installation and electrical work were not completed until September 10, when the service pump was started at 11 A.M. and the standpipe filled at noon, September 11, and the system placed in operation.

The net result of this installation in money will be the addition of one hydrant at a rental of $30 per year, the reduction of insurance rates in the sections not previously within 500 ft. of a hydrant amounting to about $100 per year for property owners, and the increase in the operating cost of the system by about $1 000 per year, including depreciation. The value of the system cannot, however, be measured in dollars and cents. If the additional fire flow will confine a small fire under adverse conditions to the starting point and prevent the loss of one life, we feel that the cost has been none too great.

DISTRIBUTION SYSTEM PROBLEMS OF THE NEW HAVEN WATER COMPANY.

BY EDWARD E. MINOR.*

[*Read April 28, 1936.*]

As I shall refer to experiences which we have had with the distribution system of the New Haven Water Company it might be well to preface my remarks with a brief description of that system. The New Haven Water Company distribution system comprises about 600 miles of pipe line supplying a territory about 30 miles in length along the northerly shore of Long Island Sound and extending northerly about 21 miles to the Town of Southington. It is in some respects a complex system inasmuch as the supply is drawn from five principal sources outside the city. These feed into a common distribution system, where they are balanced one against the other. In addition, the high-pressure system extends from west of the city through the city proper and northerly to the towns of Hamden and Cheshire. A number of high-pressure booster stations are required to supply water to higher elevations surrounding the city.

A distribution system does not start as a completed engineering plan. As the growth of the city increases the demand for water, additional resources are developed. These may or may not fit in with those already in operation. This natural growth of a water company is therefore affected not only by the rapidity and direction of growth of the city, but also by the limitations imposed by nature on resources available for the territory served. The New Haven Company has grown very much in this way for about 60 years. In 1925, it became evident that some way must be found not only materially to increase the resources, but to unify those already in service. This led to the development of our Lake Gaillard supply, with the balancing feeder running through our whole system, tying in the several sources from which water was being taken. So much for the general picture of the system.

Intakes. Formerly it was considered sufficient if the intake for a feeder line passed out into the reservoir and was submerged to a depth of 10 or 15 ft. These intakes were screened but very seldom seen. As was explained in one of our early reports, water taken from such depths would be forever pure and unpolluted. However, times change, and all of our intakes are now concrete structures having gates, so that water can be drawn from different depths, and carrying an inner wall holding fine copper screens, so that all the water is screened before passing into the pipe lines. Most of our gate wells are in duplicate so that these can be shut off and cleaned periodically. In these cleanings it was customary to find in addition to

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*Manager and Chief Engineer, New Haven Water Co., New Haven, Conn.

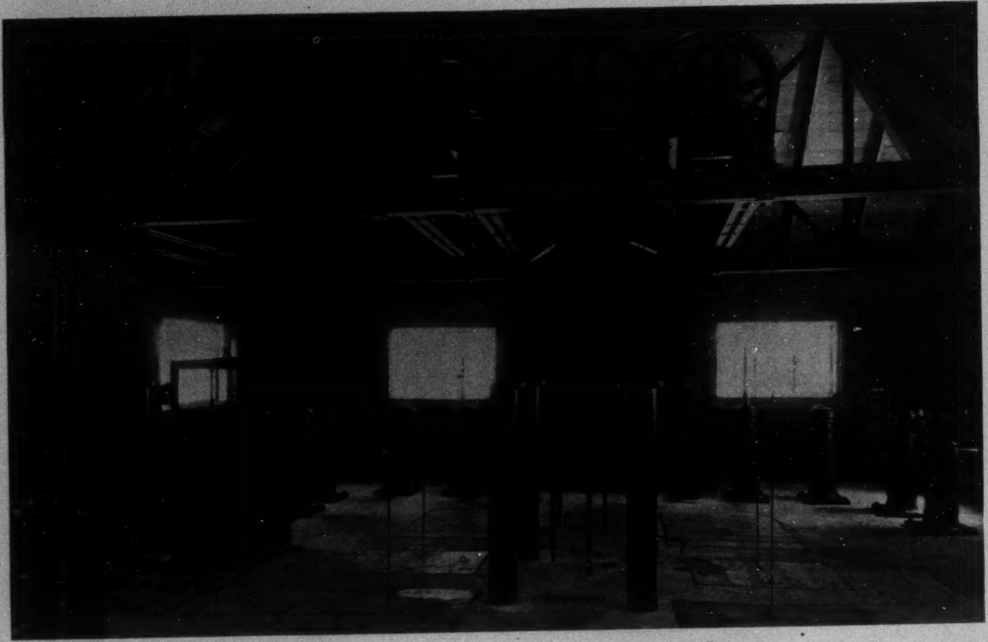

FIG. 1.— GATE HOUSE, LAKE GAILLARD, NORTH BRANFORD SUPPLY.
Overhead geared winches handling inside screens. Screens are made of Everdur. Roller bearings with copper netting and counterweighted. Screens are set in duplicate. Screen weight 800 lbs.

such material as might be floating in the water, some fish and eels. While the screens had openings not greater than ¼ in., we were troubled with eels occasionally stopping a service. These screens were cylindrical in shape and fitted into bronze channels, making a tight joint around the screen. They were located near the bottom of the well. We then developed the practice of hanging flat screens on the outside wall of the gate house over the different openings. Since that time we have had no fish or eels in the gate wells, nor any trouble in the pipe lines. Apparently a screen of the same mesh but hung on a vertical wall does not provide as good an opportunity for small eels and fish to pass through as the cylindrical basket screens located near the bottom. (Fig. 1.)

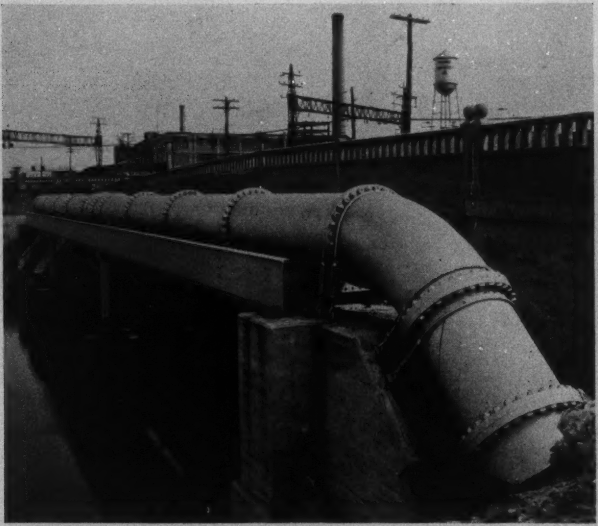

FIG. 2.— HUMPHREY STREET, NEW HAVEN.
36″ pipe crossing Mill River. Piers correspond with bridge supports. I-beam construction. Expansion joint in center of bridge.

Engineering Problems. As already noted, the development of feeder lines, secondary feeders and smaller distribution pipe usually is the result of growth and compromise. There are, however, many interesting engineering problems connected with main feeder lines, particularly on river crossings or in connection with bridges over railroads or streams. These usually require approach bends, where unbalanced pressures must be carefully watched, and also considerable provision for necessary expansion and

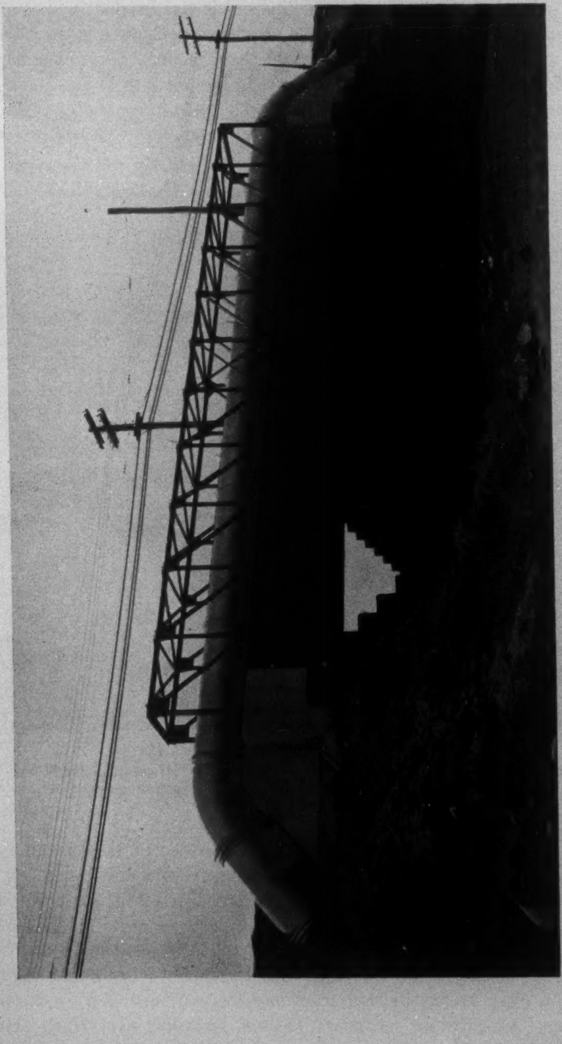

FIG. 3.— SPRING STREET, NEW HAVEN.
Pipe crossing over railroad. Through truss with pipe bends anchored to abutments.

contraction between the fixed ends of the bridge. An interesting problem is also presented where it is necessary to carry pipe lines under railroad crossings. We had one important feeder line which was carried in a concrete subway under the tracks, with suitable provision made outside the railroad right of way for relief in case of a leak in the pipe line. Some examples of these different types of crossings are given in Figures 2 to 4.

Gates. A definite policy should be decided upon in connection with the installation of gates in pipe lines. On main feeder lines this will probably be an arbitrary distance apart, modified as necessary by local conditions. We try to have control valves located on main feeders not over 2 000 ft. apart. On our 48-in. pipe line, the control gates are 42 in. and all motor operated. These gates are operated at least twice a month. This not only keeps the gate itself in good condition, but tends to keep moisture from accumulating in the motor coverings. On secondary feeders and smaller distribution pipe, we try to have each intersection controlled by at least three gates. All gates on secondary feeders, including everything above 12 in. in diameter, are gone over at least once and sometimes twice a year,

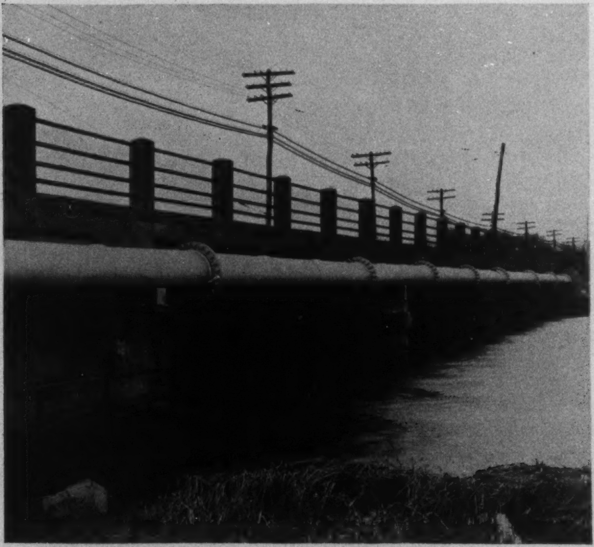

FIG. 4.— WEST RIVER PIPE CROSSING, NEW HAVEN.
24″ welded steel pipe. Expansion joints at abutments. Flanges at points of contra-flexure. Cast iron brackets. Tension in brackets taken by bolts.

partly closed and opened, and gear trains lubricated. On all gate valves operated by gears, a brick manhole is built over the operating mechanism, and a cast-iron cover, similar to that used for sewage work, is installed, thus making the gates available for lubrication or for adjustment on stuffing boxes. The manholes for electrically-operated gates are more elaborate, being provided with drains and electric light.

Flushing. A periodic flushing of the distribution system will not only tend to keep pipe lines clear but will afford an opportunity to operate and keep in repair the smaller gate valves that are necessary to control distribution. To illustrate this I might cite our experience in cutting out one section for Pitometer surveys in our city. It involved closing some 60 gate valves. The final closure brought zero pressure on the first trial. The result justified accurate records and gates that could be operated.

In flushing, avoid as far as possible limited areas. This is often necessary, but in general practice it is better to give a general cleaning at stated times, starting at the reservoir and cleaning up along the path of the general flow of the supply. As street conditions limit the amount of water one can draw and as this limits pipe velocities, it is often necessary to reverse the flow through built-up sections in order to do a satisfactory cleaning job. Also it should be noted all of such work must be handled by experienced men who can be trusted on gate operation.

Leakage. In connection with all underground distribution systems, the question of leakage is one of great importance. We feel the work of the Pitometer Company in making underground surveys for leakage and for determining rates and distribution of flow is of the greatest importance to all water companies. Extensive surveys were made by them in our system. We were fortunate in having a fairly tight system free from underground leakage. We did have, however, abandoned services,— places where excessive waste was prevalent,— and these were all graphically shown and identified in the field so that steps could be taken to prevent this loss. This resulted in the development of a system of inspection which has been continued for a number of years and has materially cut down waste. This covers about 15 000 flat-rate accounts in the city of New Haven, in addition to the metered accounts. Outside of New Haven all services are metered.

Cross-Connections. There was a time when cross-connections were regarded as being an incidental problem for water companies. Their importance was recognized, and suitable action was taken to prevent polluted supplies from this source. With the advent of air conditioning, we are faced with the fact that ground water obtained from wells is cooler in summer than water taken from open reservoirs. The ground water also is attractive for cooling purposes in connection with refrigerating machines. This at once brings into being numberless small individual wells. All of these installations are tempted to protect themselves in case of failure of the well by a cross-connection with the city supply. This at once presents to the water department a serious hazard, difficult to keep track of and limitless in its future development. As this water is usually wasted into the sewer, it may very well result in local ordinances restricting the use of public sewers for this purpose.

The development of private fire-supplies from secondary sources is not to my mind as hazardous as the practice just referred to. As these installations are always developed in connection with insurance companies, a better coöperation between the interests involved is assured. Furthermore, the danger of polluted secondary supplies connected with fire protection is fairly well known.

In addition to these, a hazard also exists where reclaimed water is used for industrial purposes. From the consumer's point of view such a practice seems to be well justified. It should, however, be guarded against as carefully as other forms of secondary supply. We had such an experience during the past year where our laboratory detected a change in the water in the distribution system. After extended tests and inspection, it was found our water was being reclaimed after being used for cooling purposes. It was then pumped into a fire system with pressure greater than the city pressure and worked out into the distribution system past a single check valve having metal seats. As soon as this situation was cleared up, we had no further difficulty with our supply.

Electrolysis. In times past, a great deal of trouble has been experienced from electrolysis, due principally to electric street-railway operation. While busses on suburban work may eventually replace the electrically-operated trolley, I do not believe they will ever meet the transportation requirements within the city. This may result in a more compact electric system where return current may not be as troublesome as in some of the more extended lines. If the condition of the track is poor or if a suitable negative return is not provided, current will shunt to the underground structures and cause serious damage both to the main pipe lines and to the services. With the increased use of alternating current in the home, other problems are now appearing where the water has noticeably been affected by the practice of grounding on metallic underground structures. This is a very important problem and one which is now being studied from the standpoint of all interests affected. It is quite desirable to have electrolysis stations located on the distribution system, so that comparative readings may be made from time to time of the extent and duration of these underground currents. In this connection, the present practice of using non-metallic joints has a very broad bearing on this whole subject. It no longer can be regarded as a large continuous metallic grid which can be used by electric companies as a negative field, without some attention being paid to its construction and dependability for the purpose intended.

Pipe Lining. In all distribution work there is always reconstruction which leaves a company with a large stock of what might be called second-hand pipe. In trying to take care of this accumulation, we have developed a routine which trims and cleans the pipe and recoats it with bitumastic enamel. This reconditioned pipe is then fully equal to new pipe. The old pipe is primed and then left stacked up until needed for use, the final lining being applied just before it is used. This resulted in such good-looking pipe that all of our new pipe is now purchased with the ordinary coating and then relined just before laying, by our own men. This is a very simple process and costs very little, but it does result in having a pipe line which is as completely protected from corrosion as it is possible to get it. So far, our examination of pipe laid has shown the coating to remain in perfect condition. Some of the early linings were affected by heat or cold, and we feel it is still desirable not to stack up lined pipe but to use it as soon as possible. The present linings are much more elastic than those first used and should result in maintaining the carrying capacity of the pipe much longer than would be possible with the ordinary dipped pipe.

Records. As a distribution system is buried in the ground, the matter of recording locations and data connected with it is of great importance. For general reference both in the office and at the yard, we depend primarily on maps. These maps are approximately 28 in. by 40 in. and show not only all buildings, street lines, etc., in the area, but pipe lines, gates, hydrants, services, curb boxes and where the service enters the building. The originals are only used for making early corrections. From these, white prints are made and bound into volumes holding about 15 maps each. When these are opened the map covers both sides of the folio. To cover our system there are about 50 of these volumes, containing approximately 900 maps. These are all drawn on the scale of 1 in. = 50 ft. Each summer during July and August, a field party picks up all new construction during the past year, all new services, and in addition, does locating work on older pipe lines which were put in before careful records were made. With a suitable index, these books provide a convenient and accurate reference for any locality covered by the company's system. These are supplemented by card files covering complaints, gate operations, pipe locations, gate locations, tap connections and other branches of work where future reference is desirable and necessary. While the initial cost of this system of records may seem high, it requires relatively little expense to maintain it, and it provides useful and valuable information. In these days when so many interests are found in the city streets, it is particularly useful to be able to show the location of various underground structures, and the exchange of information between other utilities with structures in the street is almost essential if any new work is to be undertaken.

Mechanical Equipment. I have taken the liberty of adding one more subject to this rather random paper, this being mechanical equipment. There are a number of companies supplying various sorts of mechanical gadgets for water supply work. It is quite important for the maintenance department of any water company to look into these and determine which are of real benefit in case of emergency. The different types of pumping equipment are quite important to know, so that one may feel assured, when an emergency comes, that the equipment one has can be depended upon and will deliver the goods.

Portable lighting outfits are important; pipe and gate locators; electric thawing apparatus for winter work. This can be either motor-operated or take current from existing electric lines. This whole subject offers a very interesting field for research and study. There is a good deal of satisfaction in going out on an emergency call and being equipped for any situation which may come up. With the great amount of gasoline-operated equipment now on the market, it is also quite important to have men trained in handling this equipment,— men who can find out why a gas engine will not go and who can supply the necessary touch to make it go.

REMODELED PUMPING STATION AND NEW DIESEL ENGINE, PENNICHUCK WATER WORKS, NASHUA, N. H.

BY WILLIAM F. SULLIVAN.*

[*Read September 19, 1935*].

The caption of this paper might be called "Keeping up the Reserves or Maintaining Excess Pumping Capacity."

Since 1854 various makes and types of pumps have been used by the Pennichuck Water Works. In the early days the difference between necessary and reserve capacity was small. For the past forty years the excess capacity has been maintained at a ratio of about 5, 6 and 7 to 1. Two units, which formerly were first-line pumps, have been gradually relegated to reserves and are now considered obsolete. One of these was a water-power pump; the other a steam pump. This left three active pumps maintained in prime condition ready to serve; first, a Snow triple-expansion steam-driven horizontal duplex pump of 8-m.g.d. capacity; second, a specially designed water-power pump of 6-m.g.d. capacity; and third, a 1-m.g.d. electrically-driven DeLaval Centrifugal pump which can be run from power generated at our sawmill or from the New Hampshire Public Service Lines.

It might be thought that pumps of 15-m.g.d. capacity would be ample for a city which uses less than 3 m.g.d. This would be true if the pumps were operated by steam or oil engines or driven by electric motors. The situation is this: we pump 80 to 90 per cent. of water with water power, and to get a reasonable return on capital investment, we should utilize the water storage reserves during the dry periods. At such times, with run-off diminished and the impounded water levels down, we depend on our steam pump which has been a standby unit for many years. If this pump should become disabled, repairs might take weeks, and we would be confronted with an emergency to overcome which would require costly temporary expedients, or we might find upon examination that the sound thing to do would be to put in a new installation.†

As an insurance against a contingency of this kind, as well as maintaining the policy of keeping up new equipment, it was deemed prudent and advisable to consider installing a new pumping unit. We engaged Metcalf & Eddy to make studies of the type of prime mover and pump which would best suit our conditions; that is, an installation which would be a standby capable of being operated when the water power is not available.

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*President, Pennichuck Water Works, Nashua, N. H.

†EDITOR'S NOTE: Mr. Sullivan has informed the Editor that instead of being first called into service during a drought, the new pumping unit was first operated during the great flood of March, 1936, when the low-level power station was under water.

Investigation showed that a Diesel-driven centrifugal pump would be more economical than either steam- or electrically-driven pumps. An electrically-driven centrifugal pump would show some advantage, were it not for the standby charge which, in our case, would be for approximately ten months in the year. With this information, Metcalf & Eddy prepared plans and specifications for a Diesel-driven centrifugal pump capable of pumping 6 m.g.d. The question then was, where should this unit be housed; in a separate station or located in either of the two main stations? After full deliberation it was decided that the east wing of the Snow steam station would be a suitable place. It would also simplify suction and force main connections together with economy in heating.

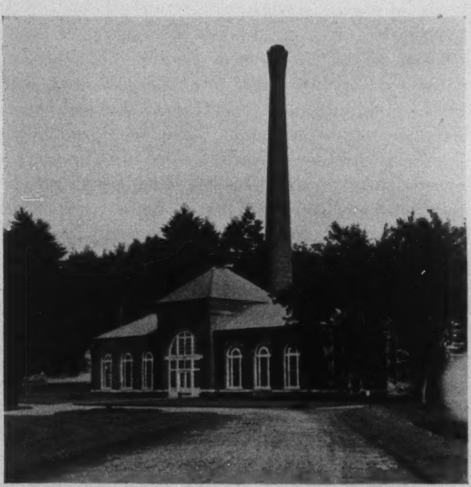

Fig. 1.— Pumping Station after Remodeling.

This wing had been used for the storage of coal. In doing away with the coal pocket, the natural thing to do was to install a fuel-oil burner under the boiler to generate steam for the Snow Pump.

The boiler plant consisted of two horizontal tubular boilers of 150-H.P. each. These replaced the original Manning boilers. As this unit is an auxiliary plant and one boiler is sufficient for running the engine, we installed oil heat only under one boiler using the same grade of oil that is used in the Diesel engine. Provisions have been made so that we can use a cheaper grade of crude oil if desired. We retained the grates under the other boiler, thus enabling the use of either coal or wood fuel. This is a precaution against a possible shortage or embargo on oil or coal. We have unlimited wood available from the marginal lands. Incidentally we saved a moderate outlay, at this time, by not investing in an additional burner.

It was decided that the Diesel engine should go into the Snow Station which was erected in 1893 under the supervision of the late John R. Freeman. This structure, as originally designed, had three sections: a central section, high with hip-roof to provide room for two vertical Manning boilers; and an east and west wing, one for the pumping unit and the other for coal. Metcalf & Eddy then prepared plans not only for the Diesel room but for a complete remodeling of the building.

Building. In making the changes, the east or coal-pocket wing of the Snow Station was practically demolished and rebuilt, and over the Snow Engine room the gravel roof was replaced with a pitched roof similar to the one over the Diesel room. The roofs were made up of steel trusses with gypsum planking and covered with Johns-Manville rigid shingles matching the slate of the original central section of the structure. Large windows were cut into the front elevation of the west wing to correspond with the east end or Diesel room. The central or boiler room front elevation was radically treated by making a nearly glass front so as to harmonize with the design of the wings. (Fig. 1.)

The new engine room is finished in buff brick with a dado of black polished Transite panelling made by the Johns-Manville Co. A Sanacoustic tile ceiling also made by Johns-Manville absorbs the noise of the engine. A two-ton travelling crane is used to simplify inspections or repairs of any of the engine or pump parts. A red brick tile floor gives a pleasing tone to the room. This entire wing has been made as fireproof as possible. The architectural changes to this forty-year old building give it the appearance of a modern and up-to-date powerhouse.

Diesel Engine and Centrifugal Pump. Diesel installations mark a forward step in New England water works practice. Installations of this type are not new to New England or elsewhere but will become more common as older steam units are replaced or when new or additional pump installations are required. This installation is unique in that it is a complete self-contained plant independent of outside power, even for lighting. While it is principally to be used as a secondary pumping unit during times when there is a deficiency of water for the main water-wheel-driven pump, it is designed for continuous duty and represents the best modern engineering practice for extreme reliability and overall economy.

The driving unit itself is a Worthington 5-cylinder 4-cycle Diesel engine of the direct-injection type, having a cylinder bore of 10¾ in. and stroke of 14½ in., normally rated at 250 H.P. at 400 r.p.m. (Fig. 2.)

Since the engine is really the heart of the installation a description of its construction may be interesting and I am largely indebted to Mr. Thomas P. Boyd of the Worthington Pump Manufacturing Co. for it.

Outwardly one sees a heavy cast-iron base on which is mounted a box frame. The cylinder heads are mounted on top of the frame and provide exhaust and air-intake connections to long manifolds which are piped as shown. The complete unit rests on a heavy concrete block about 23 ft. long, 7 ft. wide and 9 ft. deep weighing approximately 105 tons. The engine base contains the main shaft which is a solid one-piece forging 6¾ in. in diameter supported on Babbitted bearings. The bottom of the base forms a lubricating-oil sump which is connected to the suction of a gear-type pump driven directly from the main shaft. The pump delivers oil to ducts or passages cast into the base. These passages are carried up through the transverse members supporting each bearing and carry oil to the bottom of each bearing shell. The main shaft is drilled so that, upon leaving the bearings, oil continues through to the Babbitted connecting-rod boxes. The main bearings and connecting-rod boxes are split and may be adjusted by shims between the two halves or even completely removed from the engine without disturbing the shaft. The connecting rods are also drilled for passage of lubricating oil up to the wrist-pin bearings which are removable bronze bushings. In lubricating this entire assembly, which we might call the "running gear," not a single exposed pipe or fitting is used which might break or loosen in service. The oil sump in the bottom of the base is covered with a coarse-mesh strainer to exclude foreign matter which might clog or damage the oil pump. A filtering element which may be cleaned by merely turning a convenient handle once every six or eight hours is provided at the discharge of the oil pump.

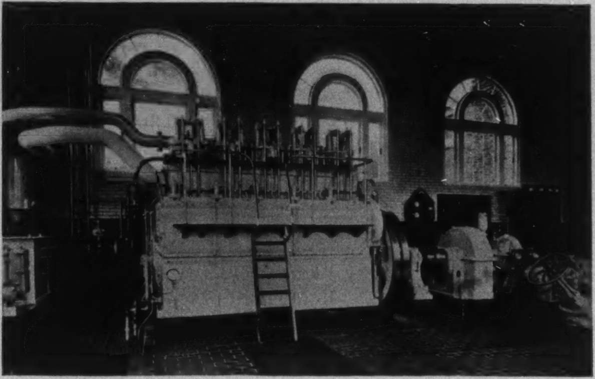

Fig. 2.— General View of the Diesel Engine.

The main frame which encloses the running gear contains the cylinder liners which are pressed in from the top and are flanged for a metal to metal seat on the top frame surface. These liners pass through the bottom of the frame where a packing box is provided. The space between the liners and the frame forms the water jacket. During operation the liner must be free to expand, which it does through the packing box. Inspection plates to the water jacket are provided for cleaning, when necessary, and light-weight oil-tight covers provide access to the running gear inside.

Mounted inside the frame on adjustable split, Babbitted bearings, pressure-lubricated, is the cam shaft driven by a gear train from the engine. The cams are hardened and ground, keyed to the shaft and locked in position. They include cams for operation of exhaust and air intake valves, fuel pumps and starting air valves, and being completely enclosed, are operating in a continuous bath of lubricating oil.

The cylinder heads, which seat directly on the top surface of the liner flange without gaskets of any kind, contain the air intake and exhaust valves, fuel spray valve and air starting valve. The head is of slightly conical shape to permit full expansion without unequal heat stresses and is water jacketed. The inlet and exhaust valves are interchangeable and are fitted with cages so that they may be removed as separate units without removing the cylinder head.

The fuel system is of the Bosch type, fuel entering the cylinders through a pressure-actuated spray-valve at a pressure of about 3 000 lb. per sq. in. Each cylinder has its own individual pump which may be removed as a unit. While the time or point in the crank cycle at which fuel injection occurs is always the same regardless of load, the time at which injection is stopped varies according to the load imposed; in other words, as the load on the engine is increased the governor responds to the demand for additional fuel by automatically lengthening the duration of injection. Similarly, as load is thrown off the fuel supply is reduced. With a compression of 500 lb. per sq. in., the temperature in the cylinder due to compression alone approximates 1 000° F. Injection occurs at about 4 degrees before top center, and during the course of combustion the pressure in the cylinder may rise to 575 or 600 lb. per sq. in. The fuel jet entering the cylinder at a pressure of 3 000 lb. per sq. in. has tremendous penetrating power and would easily penetrate a pine board ½-in. thick in the open air.

In starting the engine, compressed air from two 8½-cu. ft. storage tanks charged to a pressure of 250 lb. per sq. in. is used. These tanks are of sufficient volume to insure about five starts before recharging is necessary. Only one or two revolutions of the engine are required before regular firing begins, so that in practice the starting air gives little more than a push. Air is compressed by either of two Worthington air-cooled 9-cu. ft. compressors, one being driven by a 2-H.P., D.C. motor and arranged to maintain pressure in the starting tanks automatically; the other compressor is driven by a small gasoline engine and is intended to be used only in emergencies. Air for the engine is drawn through a Vortox air filter so placed that air can be drawn in from outside the building wall. At present, air is simply taken from the engine room. In passing through the filter, the air goes through a screen of oil vapor and fine wire mesh which prevent dust or dirt being drawn in. Exhaust gases from the engine are piped out through the building wall into a Maxim silencer so that noise is almost entirely eliminated.

In the rear of the station two 7 500-gal. fuel-oil tanks are supported on concrete cradles. These tanks are interconnected, and both are equipped with heating coils for use in cold weather. Ordinarily, no heating will be required for oil going to the engine.

From the tanks outside, oil is piped through a concrete pit into the building and connects with a small D.C. motor-driven gear-pump, first having passed through a duplex strainer. The transfer pump delivers oil to an overhead 100-gal. day tank from which fuel is fed by gravity to the engine injection pumps. Even in the coldest weather a supply of oil sufficient for about 8 hours of operation is therefore available at room temperature. A hand-operated pump may be used to fill the day tank, if for any reason the motor-driven unit does not function.

Cooling water for the engine is fed from an overhead supply tank equipped with a float valve. After circulation through the engine jackets, the water passes through sight-flow fittings visible to the operator and into an open funnel to the floor drain. By throttling the supply of cooling water, the temperature may be raised, and the piping is so arranged as to by-pass this hot water from the drain into the suction of a hot-water circulating pump which circulates through the fuel-tank heating coils.

Interconnected with the engine lubricating-oil system is a Hydroil Centrifuge, so arranged with a system of valves that it can be used for continuous filtration of oil with the engine in operation. This unit is complete with its own transfer pumps which are motor driven and can draw oil either from the engine crankcase or from either of two tanks shown beside the unit. Water and foreign matter are removed from the oil and drained to waste. The purified oil can of course be returned to the engine or whichever tank is reserved for the purpose.

Pump. The pump itself is a 10-in. Worthington single-stage double-suction centrifugal pump rated to deliver 4 200 g.p.m. or slightly over 6 m.g.d. at 160-ft. total dynamic head. The efficiency of the pump at the design point is 86 per cent., corresponding to a brake horse-power requirement of 197. It is designed to operate from rated speed of 1 350 r.p.m. up to 1 424 r.p.m., at which it will deliver about 3 500 g.p.m. at 200-ft. total dynamic head with practically no change in efficiency. The speed may similarly be reduced below 1 350 r.p.m. to a minimum of approximately 1 250 r.p.m., thus giving an extremely wide range of capacity to meet any ordinary condition imposed.

Between the pump and the engine is a Farrell Birmingham gear set connected by means of a Fast all-metal coupling. The gear is a complete self-contained unit having its own lubricating-oil pump and cooler. The purpose of the gear is to retain all the wearing qualities and reliability of a relatively slow-speed driver with the moderate proportions of a higher-speed pump. The mechanical efficiency of the gear is from 97½ to 98 per cent.

Priming of the pump is accomplished by a Skidmore automatic primer. A small chamber on the suction side of the pump contains a float the function of which is to operate a diaphragm valve regulating the flow of water through a hydraulic air-ejector.

In operation, air is removed from the pump casing and as water rises in the chamber to a level above the highest point in the pump casing, the float operates to shut off the flow of water through the diaphragm valve. In the event that air accumulates in the pump casing at any time, the hydraulic ejector operates to maintain the prime.

On the discharge side of the pump is an automatic check or Roto valve made by the S. Morgan Smith Co. This valve operates by means of pressure differential on a diaphragm, one side of which is connected to the pump side of the valve, the other to the discharge side. A drop in pressure on the pump side causes the diaphragm to operate a small piston valve which in turn operates a piston and linkage effecting closure of an exaggerated plug cock which simply rotates to either a closed or open position. The actuating medium is water velocity through the discharge main, and the rate of closing or opening is such as to avoid shock or water hammer.

Auxiliary Equipment. The station lighting-system and auxiliary motor-driven equipment receive power from a 10-k.w. Ideal D.C. generator, driven by V belts from an extension of the high-speed gear shaft. The generator also serves to charge a storage battery which floats on the line with a trickle charge while the generator is running. A Solenoid-operated relay, excited from the generator, is mounted on the switchboard and serves to switch the station load over to the battery automatically when the generator stops and the relay coils are de-energized. The battery itself consists of 60 Exide cells in sealed glass jars and has a capacity rating of 80 amp. hours. The cells are arranged in a two-tier rack enclosed in a cabinet. The switchboard is also connected in with a water-wheel-driven generator at the so-called Dean and Main Station. The battery may be charged from either this water-wheel-driven generator or from the Diesel generator.

The station is further equipped with a Venturi meter, cabinets for tools and spare parts and the necessary gages and instruments, such as engine-speed-indicator, temperature pyrometer for recording engine exhaust-temperatures from each cylinder individually and the usual gages.

Test Results. The test results on this unit indicated 4 700 g.p.m. or nearly 6.8 m.g.d. delivered at a total dynamic head of 160 ft. and speed of 1 350 r.p.m. This is about 12 per cent. capacity in excess of the manufacturer's guarantee. During the test, fuel consumed by the engine was carefully measured and found to be some 4 per cent. below the guaranteed figure. It is reasonable to assume economy will be improved as the equipment is operated and friction losses of newly machined surfaces decreased.

On a basis of delivering the full 4 700 g.p.m. at 160-ft. total dynamic head, or at the rate of approximately 6.8 m.g.d. a fuel consumption of 41 to 42 gal. per million gallons of water delivered can be expected. With commercial grades of Diesel fuel available at approximately 5½c. per gallon this is equivalent to a fuel cost of $2.25 per million gallons of water pumped. Lubricating oil consumed by the engine can be expected to run under ⅓ gallon of oil per million gallons of water pumped.

The time to raise steam in a cold boiler may take hours. Starting a Diesel is a matter of minutes; usually, if the pump is primed, it can be accomplished in seconds.

Costs. The total cost of remodeling is shown in detail below:

Building, engine foundation, drain and oil tank foundation, alterations for oil-burner installation	$16 138.00
Worthington Pump & Machinery Co., engine, pump and accessories	18 017.00
Plumbing	102.00
Pennichuck Water Works, labor and material for suction, force main and kindred work	9 562.33
Crane, Venturi meter and other accessories	4 382.16
Engineering and supervision	6 060.00
Total	$54 261.49

Osgood Construction Co. of Nashua was the Contractor for remodeling the building. Donald C. Calderwood was the resident engineer.

BROOK DIVERSION TO IMPROVE THE MANCHESTER, N. H., WATER SUPPLY.

BY PERCY A. SHAW.*

[*Read May 21, 1936.*]

Since the writer came to the Manchester Water Works in 1933, we have made a study of the source of supply to determine just what are the conditions affecting the supply and whether we could safely continue to rely on storage for our primary method of treatment, and if so what, if any, changes should be made to make this possible.

To discuss this study and at the same time describe the construction work which has so far resulted from it would make too long a paper, so I shall limit myself to describing two diversions that we have effected, without developing in detail the reasons for making them.

Spring Valley Brook Diversion.

Massabesic Lake, the source of Manchester's water supply, is a twin lake. The greater part of the inflow enters the East Pond and the water is pumped from the West Pond. The Spring Valley Brook has a watershed area of 2½ sq. mi. and enters the West Pond of Massabesic Lake half a mile north of the High Service Pumping Station. Because the city had grown out to this watershed, we decided to make Spring Valley Brook flow into the East Lake and travel a distance of about 4 miles before reaching the High Service Station; believing that the long storage secured in this way would be worth the trouble and cost. The two ponds forming Massabesic Lake are not far apart at their north ends and are separated by a low divide. The straight-line distance between the point where the Spring Valley Brook enters the West Pond to the west edge of the East Pond is about 2 000 ft. (Fig. 1.)

The Brook enters the Lake after passing through culverts under Candia Road and the Portsmouth Branch of the Boston and Maine Railroad. The West Pond, at the point where the brook enters, is shallow and swampy over a considerable area so that, in order to get the brook water to the East Pond, it is not only necessary to cut through the divide but also to build a dike across the north end of the West Pond.

A pipe line for this diversion was considered not to be practical because of the fact that so little head is available to force the water through a pipe. Any head used in this way would have to be made up by increasing the elevation of the railroad at the point where the brook crosses under it. As the flow was expected to exceed 100 c.f.s., at long intervals, the cost of a pipe at an extremely flat grade would be more than for a canal. An additional consideration would be the probability that at very low velocities a large amount of silt would be deposited in the pipe; a canal would be easy to clean.

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*Engineer and Superintendent, Manchester, N. H., Water Works.

As originally projected, the canal was to be an unlined cut through the divide with the velocity limited to 1 ft. per sec. When the preliminary layout was made, we assumed that the material would be the hardpan usual to this area. When test pits were dug, the material was found to be, for the greater part of the distance, a very fine sand. This fine sand flowed freely under the test pit sheeting. The fine material made it necessary to provide a lining for the canal and to devise methods for handling this material during construction.

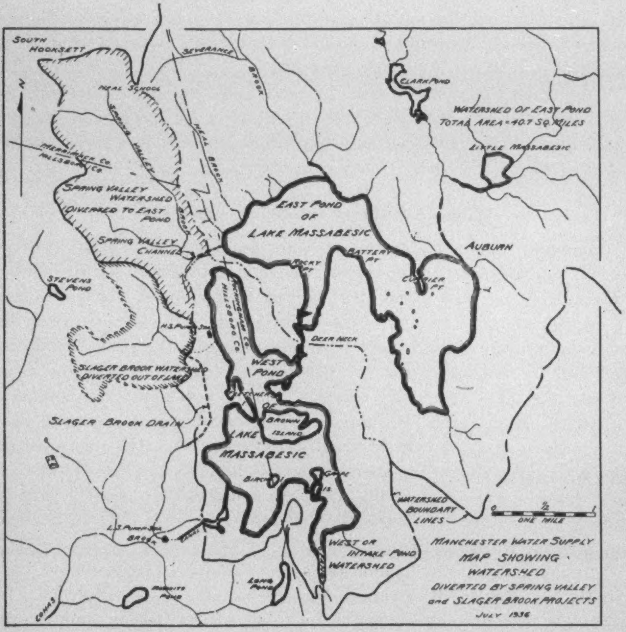
SOUTH HOOKSETT
N
NEAL SCHOOL
SEVERANCE BROOK
SPRING VALLEY
NEAL BROOK
MERRIMACK CO
HILLSBORO CO
SPRING VALLEY WATERSHED DIVERTED TO EAST POND
SPRING VALLEY CHANNEL
BROOK
STEVENS POND
H.S. PUMP STA.
ROCKINGHAM CO
HILLSBORO CO.
SLAGER BROOK WATERSHED DIVERTED OUT OF LAKE
SLAGER BROOK DRAIN
CLARK POND
WATERSHED OF EAST POND
TOTAL AREA = 40.7 SQ. MILES
LITTLE MASSABESIC
EAST POND OF LAKE MASSABESIC
AUBURN
ROCKY PT
BATTERY PT
CURRIER PT
DEER NECK
WEST POND
LAKE MASSABESIC
BROWN ISLAND
OF
BIRCH I.
GRAPE IS.
WATERSHED BOUNDARY LINES
0 ½ 1
ONE MILE
L.S. PUMP STA.
BROOK
LONG POND
WEST OR INTAKE POND WATERSHED
MANCHESTER WATER SUPPLY
MAP SHOWING
WATERSHED
DIVERTED BY SPRING VALLEY
and SLAGER BROOK PROJECTS
JULY 1936

FIG. 1. — PLAN SHOWING WATERSHEDS DIVERTED.

Surveys showed that the canal would be about 2 050 ft. long. Of this 600 ft. would be through low swampy ground below high-water level and 1 450 ft. through a low sandy divide between the ponds. The maximum cut for the canal would be about 11 ft. At the east end of the project, the canal water would have to flow under the Londonderry Turnpike; so at this point a culvert about 100 ft. long was designed to discharge the water from the canal to the Lake.

The 1 450 ft. of canal to be excavated through solid ground, were given a width of 5 ft. at the bottom with two-on-one slopes on the sides. The bottom and sides of the canal, to a point above high water level, are lined with field stone placed as paving. This is generally from 8 to 12 in. thick. The job took 1 400 cu. yd. of paving stone and used up plenty of stonewalls. The greater portion of this had to be fished out of snowbanks 3 ft. deep. Above the paving, the banks were covered with the top soil stripped from the canal area.

Through the swampy area, where the canal would be in muck with sand below, we planned to blast the canal to a width of 10 ft. on the bottom, so as to allow for sloughing of the muck on the sides. A trench blasted in this sort of material generally has one-on-one side slopes, unless there is a tough surface mat, in which case the sides will be practically vertical.

With 100 c.f.s. flowing through the diversion channel and the lake full, we expect a loss of head of less than half a foot in the culvert and canal. At full Lake the canal has a water area of about 75 sq. ft. The culvert is of reinforced concrete, 5 ft. wide and 4 ft. high. The railroad bed is low at the culvert with rather steep grades in both directions from the culvert. We offered to pay for raising the railroad grade a maximum of 2 ft. for a distance of about 900 ft. provided that they would at least double their culvert capacity. The present 3-ft. culvert is entirely inadequate for exceptional flows, although it takes the usual spring flood without difficulty. The railroad agreed to this proposal.

The dike across the end of the pond is 1 730 ft. long and is made from the material taken out of the canal section. This material is not well suited for dike construction, but as the height is not over 8 ft. and usually much less and as the width is actually about 12 ft. on the top with flat side slopes, it will undoubtedly be permanent. It will be protected on the Lake side by stone riprap taken from the Auburn pipe job. The Spring Valley Channel and dike are shown at the north end of the West Lake in Figure 2.

We had our information and plans ready last summer and applied to the WPA for funds. The local office approved the project and sent in a request for approval to Washington. Approval was long delayed due to some changes in the WPA Program and when they were finally ready to start, we told them it was not the time in the year to do the work. However, employment was needed for the very large number of men on WPA rolls, and they insisted that we proceed through the winter. The work was started December 1. The weather turned intensely cold almost immediately and continued unusually cold until it suddenly broke early in March. We started work on the culvert, and at the same time started excavating at the east end of the channel. We began the excavating work with a small steam shovel rigged as a drag-line. Good progress was made for a short distance where we had a sand formation under-lain with rock flour. After about 200 ft. of this material, the lower portion of the cut changed to a fine cemented gravel which was too hard for the light drag line to handle. The

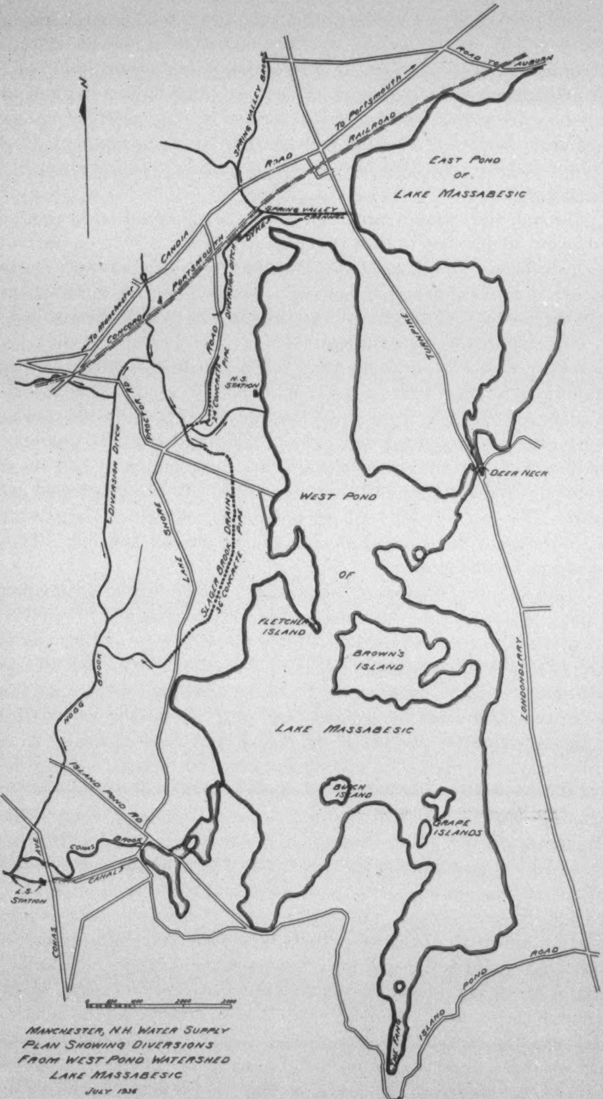
N
ROAD TO AUBURN
To Portsmouth
RAILROAD
ROAD
SPRING VALLEY BROOK
EAST POND OF LAKE MASSABESIC
SPRING VALLEY CHANNEL
DYKES
CANDIA
To Manchester
CONCORD & PORTSMOUTH
DIVERSION DITCH
ROAD
24" CONCRETE PIPE
N.S. STATION
TURNPIKE
PROCTOR RD
DIVERSION DITCH
DEER NECK
WEST POND
LAKE SHORE
SLAGER BROOK DRAIN 36" CONCRETE PIPE
OF
FLETCHER ISLAND
BROWN'S ISLAND
HOGG BROOK
LONDONDERRY
LAKE MASSABESIC
ISLAND POND RD
BIRCH ISLAND
GRAPE ISLANDS
AVE
COHAS BROOK
CANAL
L.S. STATION
COHAS
0 500 1000 2000 3000
ROAD
POND
ISLAND
THE FANG
MANCHESTER, N.H. WATER SUPPLY
PLAN SHOWING DIVERSIONS
FROM WEST POND WATERSHED
LAKE MASSABESIC
JULY 1936

FIG. 2. — LOCATION OF DIVERSION STRUCTURES.

rig was then changed to a shovel, which made fair progress until the ground froze about 18 in. deep over one of the long WPA week ends. The shovel was unable to break through the frost or to handle the material to advantage when blasted.

At the request of the WPA Supervisors, the shovel was taken out, and 75 additional men put in its place. We had been using men to handstrip the top soil but found that when we got ahead of the excavation with the stripping it merely encouraged the frost to enter the ground more deeply. We then started the practice of grubbing and loosening the soil and leaving it in place until just before excavating.

Knowing that after 500 or 600 ft. of firm material had been excavated we would run into the flowing sand area, we laid a pipe drain under the bed of the canal with the expectation that we might have to go through the sand area first with a sheeted ditch to drain the sand before we could excavate the canal. In order to avoid this if possible, we started a draining process when work was first begun. At a point about the center of the sand area we carried one of the test pits down about 4 ft. below the canal grade and put in a 3-in. centrifugal pump. This small pump, which was arranged to stop and start automatically, was kept in operation 24 hours a day from early December until the job was finished in April. Later, another pump was added farther along the trench. This long-continued pumping gradually drew the ground water down, so that we found it possible to excavate down from the surface, shovelling the material into trucks.

The continued cold helped us in two ways: when we went down through the fine sand, the slopes froze solid and our dike, which would have been impassable for trucks, froze as fast as it was placed. At night we pulled a heavy drag over the new material, and in the morning we had a hard, smooth-surfaced highway. On the other hand the cold was so intense that the material being excavated froze from 2 to 8 in. over night, and one crew was kept blasting the surface ahead of the excavating work all through the center part of the job.

For a distance of about 800 ft. the dike was built through several feet of muck in the swampy lake bottom, and as we wanted the fill to rest on solid bottom, a channel was dynamited in the muck ahead of the fill. The weather was so cold that it was necessary to blast only so much channel as could be filled in half a day, or the broken ice resulting from the blasting froze so hard that it would have to be broken again before material could be placed. When the weather warmed up and the flood came in March, the water flowed under our dike fill in two places; but otherwise the embankment held and seems to be firm and solid. It has not settled as much as some of us expected.

Early in the progress of the job, we dynamited the channel from the railroad culvert to the point where firm ground was encountered. This was done by pushing a pole into the soft material and inserting "ditching" dynamite. To make a ditch 3 ft. deep with a 10-ft. bottom, required 1.5 lb.

of dynamite per foot or practically a pound per cubic yard. The company from whom the dynamite was purchased advised us as to the exact location and depth of holes and amounts of dynamite. Their plans worked to perfection, and we used almost exactly the estimated amount of dynamite and secured a very satisfactory trench.

We started paving the slopes of the canal as soon as they were excavated, but as the slope froze solid it was necessary either to pick the slope to fit the stones or to excavate below grade and fill back with sand in order to secure a bed for the stones. This made paving extremely slow and expensive. The greater portion of the paving was placed in the three or four weeks after the weather began to moderate. When the Lake goes down and the brook practically dries up in August, there will be some trimming and paving to do at the junction of the excavated and blasted sections. In spite of the intense cold and being flooded out by the high water, we secured what appears to be a very satisfactory job.

The railroad has not yet made its changes as its men have been too busy mopping up after the recent flood.

The total cost of the work not including railroad changes was $55 000. We believe it could have been constructed as a WPA Project in the fall of the year at a cost of little over one-half what it cost in the winter. However we kept about 175 men busy for nearly four months. The cost to the Manchester Water Works was $9 300; as we had estimated that the job would cost about $22 000 by contract, we got out of it fairly well at that.

SLAGER BROOK DIVERSION.

The Slager Brook has a watershed area of about 700 acres and a population of 568. It drains an area in the eastern part of the city of Manchester which has, in recent years, been built up to a considerable extent but in which the street surfaces are unpaved and in which there are practically no sewers. This brook unfortunately enters Massabesic Lake within 300 ft. of the High-Service Pumping Station. Tests of the brook water at various times have always showed it to be rather badly polluted. About one quarter of the total area is swampy or marshy, making the color of the water very high. The contamination is principally from horses, cows, goats and pigs, but the water is certainly undesirable for drinking purposes.

In 1923, at the suggestion of the writer, the upper half of this brook was diverted, by means of a ditch, out of the Massabesic Lake watershed into Hogg Brook, a tributary to Cohas Brook at a point below the entrance of the Lake water. In 1933, the ditch was found to be caved in and not in use; it was immediately cleaned out and has since been discharging into Cohas Brook.

After a considerable amount of surveying and inspection, it was found possible to divert the remainder of the Slager Brook flow out of the Massabesic Lake watershed into the same small brook as the upper portion, but at a different point. The surveys showed that, from the highest point at which the brook could be satisfactorily intercepted to the outlet of the swamp tributary to Hogg Brook, there was a drop of 6 ft. and the length of the diverting drain would be about 6 000 ft. This required a grade of only 1 ft. in a thousand, and therefore a large pipe in proportion to the amount of water to be carried. Figure 2 shows the location of the Slager Brook drains.

It was decided that considering all the circumstances, it would not be necessary to divert all of the water at times of maximum flood flows in the brook. We therefore determined to use a 3-ft. diameter drain which would have a capacity of about 30 c.f.s. or about 60 c.f.s. per sq. mi. of watershed. The intake to the drain was built with an overflow weir, so that flows above the capacity of the drain would be discharged down the old brook bed to the Lake. We planned that when such an overflow occurred no pumping should take place for at least two days after this overflow into the Lake. Further it was considered that when a flow as high as 60 c.f.s. per sq. mi. had been reached, the watershed would be pretty well washed and the flow which followed would not be of as undesirable a nature as the first flush.

Above the intake structure, a small dam was built which is provided with a 20-in. pipe outlet near the bottom in addition to the spillway. This pipe outlet is controlled by a valve. The pond formed by this dam acts as a settling basin to prevent coarse material from entering the drain. It will also have some effect in reducing the peak flow of quick flushes of water. The pipe outlet is adjusted so that all ordinary stream flows are discharged with only a slight retention. When the flow increases above normal run-off, the water builds up behind the dam and flows over the spillway.

The surface drainage of an area to the west of Lake Shore Road, and tributary to the brook below the drain intake, has been brought into this drain by a 24-in. branch running 1 600 ft. north along Lake Shore Road. This area comprises about 75 acres of land which has been built up with small scattered houses.

The inlets on the branch are provided with grates and sand traps. The brook entrance to the drain is screened with a bar screen. This screen is kept clean by the men at the nearby Pumping Station. A short distance below the intake another small brook containing road drainage is picked up. Just below this point there is a grit chamber designed to catch coarse material entering the drain at any point. This chamber is arranged to be flushed into a swampy area.

The surveys showed that the first part of the line would have to run through exceedingly rough country, and a rather curved alignment was adopted to prevent cutting through a number of glacial hillocks. Then to get out of this brook watershed, it was necessary to cut through a 16-ft. ledge barrier. From this point on, the line passes through an area draining directly to the Lake composed of small watersheds separated by rocky ridges. This means that we encountered a good deal of ledge excavation, there being ledge in some portion of the trench for about 50 per cent. of the total length of the drain. The line was curved, rather than broken at manholes, and manholes were inserted not over 600 ft. apart.

Excavation was largely by hand labor in order to give employment. A portion of the work was done by the ERA. However the ERA folded up at a most inopportune time leaving us to do a great deal of the work which they had proposed to do. The WPA was organized in time to do some of the backfilling.

The pipe used was 3-ft. reinforced concrete pipe with taper joints. This pipe was made in Merrimack, N. H. and delivered on the job at a price of $3.50 per foot. The pipe was placed with a light steam shovel rigged as a crane. Rapid progress was made on the pipe laying, the maximum length placed in one 8-hour day being 520 ft. This was double what we had anticipated.

The extraordinary flood of this year gave this diversion project a very thorough test. Instead of having to wait for five or ten years to see how our overflow worked, it was worked nearly to the limit this year. In the March high water, the flow of the brook we computed to be 54 c.f.s. or 94 c.f.s. per sq. mi. The flow in the drain was 33 c.f.s., and the rest of the water flowed to the Lake. To set the stage for this flood, we had about 30 in. of heavy snow on the watershed, hard frozen ground, 1½ in. of warm rain and warm weather combined. Flows of several cubic per second occurred where, in an ordinary spring, there would be only a trickle.

We did not suspend pumping as we had planned when the overflow occurred, because Spring Valley Brook was still discharging into the Lake and there was so much wash from other sources that it did not seem to be worth while. Instead, we increased our chlorine dosage, and our water samples from the system showed up better than usual during the period of the flood flows.

The cost of this diversion was about $65 000 of which $25 000 was for the pipe and other materials. The work could have undoubtedly been done more cheaply by the use of more equipment.

FROST PENETRATION AS AFFECTED BY WEATHER AND SNOW CONDITIONS.

BY HARRY U. FULLER.*

[*Read December 19, 1935.*]

Those among us who have been in the waterworks business for a considerable number of years have noticed that there are occasionally very cold winters. The winter of 1903, the winter of 1917 and 1918, and again the winter of 1933 and 1934, were seasons in which unusual numbers of water pipes were frozen.

The question is raised in one's mind whether anything can be done that will assist in making the life of the waterworks superintendent more pleasant during such occurrences. It would seem that if we could find a way by which we could foretell, even a short time in advance, abnormal freezing conditions, we might be able to make some preparations. I have no doubt this subject has been discussed by many in the past, and considerable has been written concerning it. In the bad year we had in 1917–1918 I made some attempt to measure the amount of cold weather, and again last year I took the matter up and carried it a little farther. Perhaps some of the information I gathered might be interesting.

Frost Penetration. At a depth of 30 ft. below the surface of the ground, the temperature range during the year is less than 0.5° F. so for our purpose we can call it constant at about 50° F.

The air in Portland has a range of temperature for short spaces of time from about 30° F. below zero to about 100° F. above zero. Taking the reports of the Weather Bureau, we find that they give the mean daily temperature. This is half way between the highest and lowest temperature for the day. The mean temperature for each day during the 60 years that the Portland Bureau has been maintained has been averaged to give what is called the normal temperature for each day in the year. These normal temperatures range from 21° F. on January 25 to 69° F. on July 25. The half-way points in the spring and fall occur on April 22 and October 27, which are nearly half way between the warmest and the coldest days. The number of days warmer than 45° F. is the same as the number of days colder than that temperature.

The forces that we have to consider are the temperature of the sun from above varying greatly with the time of year and slightly with the particular year, and the temperature of the earth below. There is a natural law that temperatures, like water, constantly try to seek a level. Heat units are said to pass from a warmer to a colder substance. The rate of flow of heat units is controlled by the resistance of different substances to that flow. It has been found that heat passes through different substances at various rates. I have never been able to figure from any of the information in text books how fast frost penetrates the ground in the Maine climate. When there is only a little frozen ground at the surface, the ground offers considerable resistance to the effect of cold weather; but when the water in the soil has turned to ice, the resistance is much less, and the cold travels down to the pipes more rapidly. This explains why it takes December and January for the ground to freeze 4 ft. deep, and then a little extra cold will freeze pipes rapidly in February.

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*Chief Engineer, Portland Water District, Portland, Me.

Snow Blanket. After a fresh fall of snow which increases the snow blanket on the ground, the warmth or rain which may follow will decrease the snow blanket until another snow fall occurs to again increase it. The snow blanket varies through the winter considerably. At Portland, the average normal snow blanket increases to about 9 in. in February and then decreases. The normal snow blanket is about ½ of the normal snow fall, which reaches 20 in. in February. The snow blanket has been at different times anywhere between 0 and 40 in.

Frost Line. Beginning about December 1 in the Portland climate, the ground begins to freeze, and the frost line or the bottom of the frozen ground goes down. The top inch of the surface of the ground is probably for any day about the same temperature as the average temperature of the air. As the winter progresses, the average temperature of the air is below 32° F. or freezing. When the average temperature is colder than 32° F., the frost line goes down; when warmer than 32° F., the frost line tends to rise from the effect of the warm earth below. These movements are affected by the snow blanket on the ground and the insulating value of the ground itself.

Temperature Deficiency. The accumulated deficiency in degrees below freezing can be found by subtracting the mean temperature for the day from 32° F. and beginning December 1 adding these deficiencies to that of the day before. It will be found that this number will increase irregularly up to the middle of March and then decrease to zero in May. At Portland, the normal average deficiency for 60 years reaches 750° F. in the middle of March and decreases to zero in the middle of May. In 1918, the figure reached 1 500° F. in the middle of March, and last winter the high point was 1 150° F. These figures are interesting because they have some relation to our experience with frozen water pipes.

A Normal Winter. Fourteen out of fifteen winters do not cause unusual trouble. The pipes in Portland are laid in a 5-ft. trench. Apparently the frost goes down nearly to the pipes, but not quite. The temperature deficiency may reach 1 100 or 1 200° F. with sufficient snow blanket, or trouble could come with a deficiency of perhaps 600° F. with practically no snow blanket. The normal winter reaches a deficiency of 550° F. on February 9, but has a normal snow blanket at that time of 8 in. These figures give some indication of the outline of our problem.

Experience in Canada. Ross L. Dobbin says, "Ottawa and Montreal report frost down 7½ ft. where 6 ft. is the usual depth. Other cities in Ontario report depths from 1 to 2 ft. lower than usual. Many Canadian water works report February 8 as the peak day."

Conclusion. We were not satisfied with the information that we were able to get from the literature and are carrying on this winter an experiment which we think will be interesting. We have inserted thermometers in pipes at 6-in. differences below the surface of the ground down to 5½ ft. The ends of these pipes are in a wooden meter box. There is a series of pipes on one side of the box which are in gravel, and a series on the opposite side of the box which are in clay. In each pipe is a thermometer, and at intervals of about one week we read these thermometers. We expect to be able to learn from this experiment the rate at which the frost line goes downward in the first half of the winter and the rate at which it rises in the last half of the winter. It was interesting to note that when we knew as a matter of fact that the ground was frozen one foot deep in a trench last week, our thermometer, which was one foot below the surface, for the first time read 32° F.; and each of the thermometers at various depths below the surface has been showing a regular decline in readings since they were installed in September.

DESIGN OF BILLS BROOK DAM, BARKHAMSTED RESERVOIR.

BY WILLIAM DORENBAUM.*

[*Read April 28, 1936.*]

History. In 1925, the attention of the Board of Water Commissioners was called by the Manager and Chief Engineer, Caleb Mills Saville, to the fact that with the prevailing rates of growth in demand, and under conservative estimates, the present supply sources would reach their safe limit within about fifteen years, and that therefore the city of Hartford should be actively and constructively taking steps towards the acquisition of additional sources of water supply for the purpose of safeguarding the future. Quoting from Mr. Saville's report, "Sources adequate for greater Hartford to at least the end of the present century are now available and immediate steps should be taken to acquire control of them."

By the year 1927, the Board of Water Commissioners had already begun to acquire land in the upper portion of the East Branch of Farmington River in Barkhamsted and Hartland. The following year, it was reported that plans and investigations for an additional supply works should be undertaken at once and that one of the factors which might be of great effect upon the proposed development was the matter of agreement with the mill owners on the Farmington River below New Hartford. It was recommended (1) that immediate steps be taken to obtain authority from the General Assembly in 1929 to use the East Branch of Farmington River immediately north of the Compensating Reservoir as a source of additional water supply for the city of Hartford; (2) that when authority was obtained provision be made for immediately proceeding with plans for the construction of the reservoir.

Since investigation and design must precede any construction work, authorization was asked for making preliminary investigations, such as borings, for the dam location. Accordingly, some wash and core borings were made, and although not sufficient to exactly locate the site of the dam, they were sufficient to indicate that the location was practical for a dam to be constructed without any unusual difficulties.

In 1929, the Legislature failed to grant the authority for the additional supply work but did pass an act authorizing the cities of Hartford and New Britain to enter into contracts concerning water supply and also an act creating the Metropolitan District. However, that year found the matter of land-purchasing progressing nicely, and at the Bills Brook Dam site two test pits were dug on the east side of the valley and one on the west side and soil samples were studied and found satisfactory by Prof. Charles Terzaghi, formerly of the Massachusetts Institute of Technology; also a topographical survey was made of the dam site.

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*Designing Engineer, Water Bureau, Hartford Metropolitan District.

In 1930, arrangements were made with R. H. Randall & Co., Inc., of Toledo, Ohio, to make preliminary surveys of the East Branch valley, including traverses, levels, staking flow line and mapping roads, fences, wooded areas, etc., within the reservoir area and 200 ft. beyond. The Water Bureau engineers began property surveys and continued the test-pit work.

At the 1931 session of the General Assembly, the following legislative acts were approved:

(1) Authorizing the Metropolitan District to store and divert water from the East Branch of Farmington River for water supply purposes and to construct dams, reservoirs, and the necessary works on the East Branch for this purpose; confirming the terms of an agreement made with the Riparian Company as to the manner in which the riparian owners were to be compensated for the loss of water diverted from the East Branch and providing for the acquiring of land by condemnation, if necessary, and for the layout of relocated highways, removal of existing cemeteries, town buildings and bridges.

(2) Authorizing the District to supply certain towns and cities with water.

(3) Authorizing the District to furnish water to New Britain.

In November, 1931, the electors of the District voted to appropriate $8 600 000 for the additional supply development.

During the year 1931, a contract was awarded the Schlumberger Electrical Prospecting Methods for making an electrical-resistance survey of bed rock conditions at the proposed Bills Brook Dam site. This work was performed during August and September. During the year 1932, an area of about 250 acres at the dam site was cleared for the purpose of enabling a better view of ground conditions to prospective bidders on the dam construction and as an aid in studying the geology of the site. Also, an extensive set of borings was made at the dam site. The results checked very closely the results of the Schlumberger investigations. A detour road was constructed at the east end of the dam site along Bills Brook valley to bypass traffic during the construction of the dam, since the existing state highway from New Hartford to Barkhamsted, just east of the river, crossed the dam at a point where the principal early construction was involved, such as the stream control works.

In 1933, construction of the dam was started with the contract for the lowest section, including cofferdams and a conduit for stream control works. This work was completed by the spring of 1935, and a second contract was let carrying the dam construction to elevation 520, or 25 ft. below the top. This work is now in progress and will probably be completed in the fall.

The third step of construction just approved by the Water Bureau includes principally the completion of the dam, the construction of the spillway, the upper and lower gate chambers to house the control gates and power plant, and piping in the conduit or tunnel between the buildings; the clearing of the bottom 75 ft. of the reservoir basin; and the construction of the highways on the east and west sides of the valley to replace existing roads to be flooded. This work will probably be completed in 1938.

The fourth step, as at present contemplated, will be the construction of about 7½ miles of 48-in. pipe as a supply main connecting the new Barkhamsted Reservoir with the present Nepaug Reservoir. This work will probably be done in 1937 and 1938. At this time the program will have been developed to the point where it will be possible in case of necessity to draw water for use of the District.

The fifth and final step, to be undertaken in 1938 and 1939, includes the construction of about 6 miles of state highway around the north end of the reservoir to replace the present trunk line across the valley in Hartland, known as Route 20; the clearing and grubbing of the top 40 ft. of the reservoir basin; special treatment of the areas of shallow flowage; and the cemetery removals.

It is proposed to start filling the reservoir in 1938, and this probably will be accomplished by 1941. This seemingly long period for filling a 30-billion gallon reservoir is due to the fact that in accordance with the terms of the agreement with the Riparian Owners, the filling can be done only from so-called flood waters, which in this case are flows in excess of 2 c.f.s. per sq. mi. from the tributary watershed area of 53 sq. mi.

Preliminary Investigations. A very extensive and exhaustive study and groups of tests of surface and subsurface conditions at the dam site were made preliminary to the design of the dam. This exploratory program included wash borings, for determination of depth and quality of overburden; core borings, for determination of the character of the rock floor; test pits, for determination and check on quality of overburden and ground water behavior; geophysical survey, for determining approximate contour of the rock floor; clearing of the land, for the purpose of exposing much better the peculiar surface features of the site and as an aid in studying their significance; detailed field examination of the geologic features and rock formations and other materials; and laboratory tests on materials, such as specific gravity, permeability and mechanical analysis. The object of all this exploratory work was to arrive at conclusive answers based on sound results and logical arguments in the following matters:— the physical feasibility of building an earth dam at the Bills Brook Dam site to hold water to at least elevation 520, the suitability of the structural materials available locally, the proper type of earth dam and economic location, the necessity for a core wall and for grouting of the rock foundation.

The first step in this elaborate program consisted of a preliminary line of 8 wash borings and 7 core borings, on a location across the valley selected by judgment of surface conditions choosing the narrow section where the valley just upstream broadens out, and a few test pits on both sides of the valley. The results of these indicated that bed rock did not follow surface contours and that there was a fairly heavy overburden. It was decided, therefore, to study the shifting of the site either up or downstream to obtain better conditions. For this purpose the geophysical survey method by electrical resistances as performed by the Schlumberger Electrical Prospecting Methods was resorted to. The entire dam site area of approximately 62 acres was investigated, requiring the taking of over 10 000 electrical measurements at about 243 stations. The investigations revealed a condition entirely unlooked for from surface investigations, and instead of a ridge with a rock core between Bills Brook valley and the East Branch proper, it appeared that the ridge was entirely composed of hard, packed, glacial drift, with a rock depth of from 80 to 100 ft. below the ground surface, and some 70 ft. below the proposed reservoir level.

The results were plotted in the form of a rock contour map. This map was then used to determine the best and most economical location, and a systematic layout of borings was made. In all, 6 wash borings and 45 core borings were drilled, and about 20 test pits were dug, 8 of them to rock. These borings indicated a depth to rock varying from 11 to 148 ft. and averaging about 40 ft., as compared with an average of 39 ft. by the Schlumberger Method. The electrical survey disclosed a deep pre-glacial gorge at the extreme east end of the dam site under the divide between the East Branch and Bills Brook valley. Under usual methods of exploration, this condition would probably have not been revealed. The borings confirmed the existence of the deep rock floor, in fact it was actually located 38 ft. deeper than was indicated by the geophysical survey. An extensive set of tests was performed to determine the permeability of the glacial drift in this gorge, also its stability and ground water movements through it.

A series of 38 additional test pits was dug in the proposed reservoir area for about 1½ miles upstream from the dam site for the purpose of determining the suitability and quantities of materials for an earth dam and the availability of proper concrete aggregates.

The results of these investigations led to the following conclusions:

(1) As a foundation support, the rock floor is everywhere sufficiently sound and substantial to carry the structures proposed.

(2) The character of the rock indicates the necessity of a thorough grouting program.

(3) The wide range in the degree of permeability of the overburden indicates the necessity of providing a cut-off trench and core wall through the main section of the dam, with provision for proper seating in sound rock.

(4) All parts of the overburden are substantial enough to serve as support for an earth dam and sufficiently stable so that there is no danger of weakening or removal or modification due to the new conditions to be imposed by the reservoir.

(5) The acceptance, as a natural extension of the main dam, of the ridge of glacial drift between the East Branch and Bills Brook valleys is safe and practical construction, because of its compact and impervious nature and because of its extent forming a barrier thicker than the section of the artificial embankment.

(6) The rolled earth type of embankment is the most desirable type of dam for this location.

The conclusions just stated were confirmed by Prof. Charles P. Berkey, Geologist of Columbia University, who personally visited the site, examined the rock cores and soil samples, reviewed the studies made by the Bureau engineers, and himself thoroughly studied conditions before submitting his reports.

Prof. Charles Terzaghi was called on, two years before, to investigate this site. He also personally visited the site, examined the test pits and made soil tests. His report compares in detail quite favorably with the later reports of Dr. Berkey.

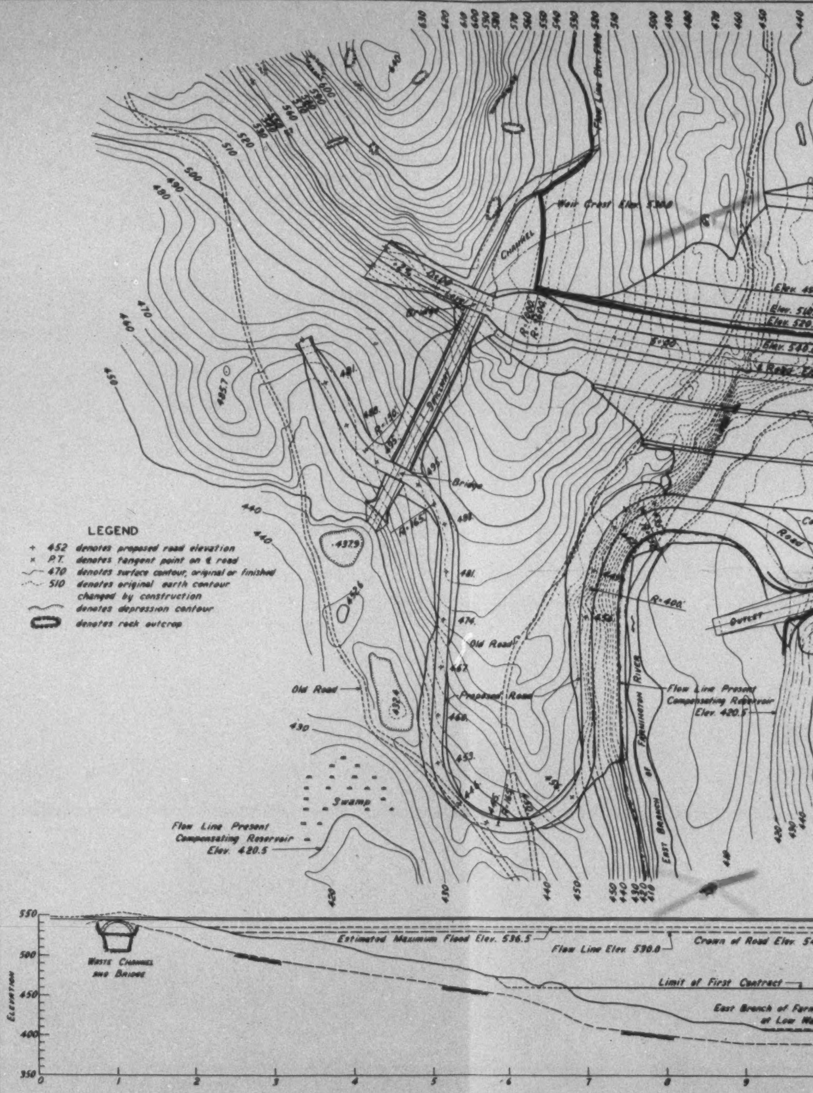
DESIGN.

Bills Brook Dam. The dam (Plate I), which is a rolled earth embankment with a concrete core wall founded on sound rock, is in plan curved downstream to a center line radius of 3 600 ft., in order to form an attractive and more natural blending with the existing topography. It is located on the East Branch of Farmington River just above the mouth of Bills Brook. The principal dimensions are:

	Ft.
Length	1 950
Maximum base thickness	900
Maximum height above river bed	137
Maximum height above rock foundation	157
Thickness at flow line (El. 530)	131
Freeboard	15

Considerable thought and study were devoted to obtaining the best section of earth embankment to be built of local materials, considering all the elements necessary for a safe as well as economical structure. The section finally adopted is shown in Plate II and may be briefly described, as follows: Upstream slopes vary from 1 on 4 at the bottom to 1 on 1½ at the extreme top. Downstream slopes vary from 1 on 2½ at the bottom to 1 on 1¾ at the extreme top with berms at three levels to care for surface drainage. Both upstream and downstream toes abut against heavy rock fills with outside slopes of 1 on 1½.

The outer faces of the rock fills are covered with well-compacted impervious soil to a slope of 1 on 3 faced with riprap or paving. These rock fills and facing serve as anchors for the dam and as cofferdams for control of the river during construction of the dam. The concrete core wall is 12 ft. thick at the bottom and is of uniform thickness for the first 25 ft. above the river bed and for the 18 ft. below to the foundation. Above this it tapers on 20 on 1 batter on both sides to a thickness of 2½ ft. at the flow line, El. 530, with this uniform thickness to the top of the dam. This wall is carried on a concrete apron spread 8 ft. upstream and 3 ft. downstream from the wall, the apron extending a minimum of 18 in. above the rock and a minimum of 30 in. into the rock with a 3-ft. cut-off below it. All weathered, disintegrated and loose rock is removed from the trench, which varies from 2 to 42 ft. in depth, averaging about 20 ft. The upstream cut and the bottom 8 ft. of the


LEGEND
+ 452 denotes proposed road elevation
× P.T. denotes tangent point on ℄ road
470 denotes surface contour, original or finished
510 denotes original earth contour changed by construction
denotes depression contour
denotes rock outcrop
Flow Line Elev. 530.0
Weir Crest Elev. 530.0
Channel
Bridge
Spillway
R-1000
R-800
R-190
R-165
R-400'
Old Road
Proposed Road
Swamp
Flow Line Present Compensating Reservoir Elev. 420.5
Farmington River
East Branch of Farmington River
Outlet
Road
Elev. 540.0
Elev. 520.0
Waste Channel and Bridge
Elevation
Estimated Maximum Flood Elev. 536.5
Flow Line Elev. 530.0
Crown of Road Elev. 5
Limit of First Contract
East Branch of Farm
at Low W
550
500
450
400
350
0
1
2
3
4
5
6
7
8
9

LONGITUDINAL SECTION ON CENT

SCALE 80 20 0 80

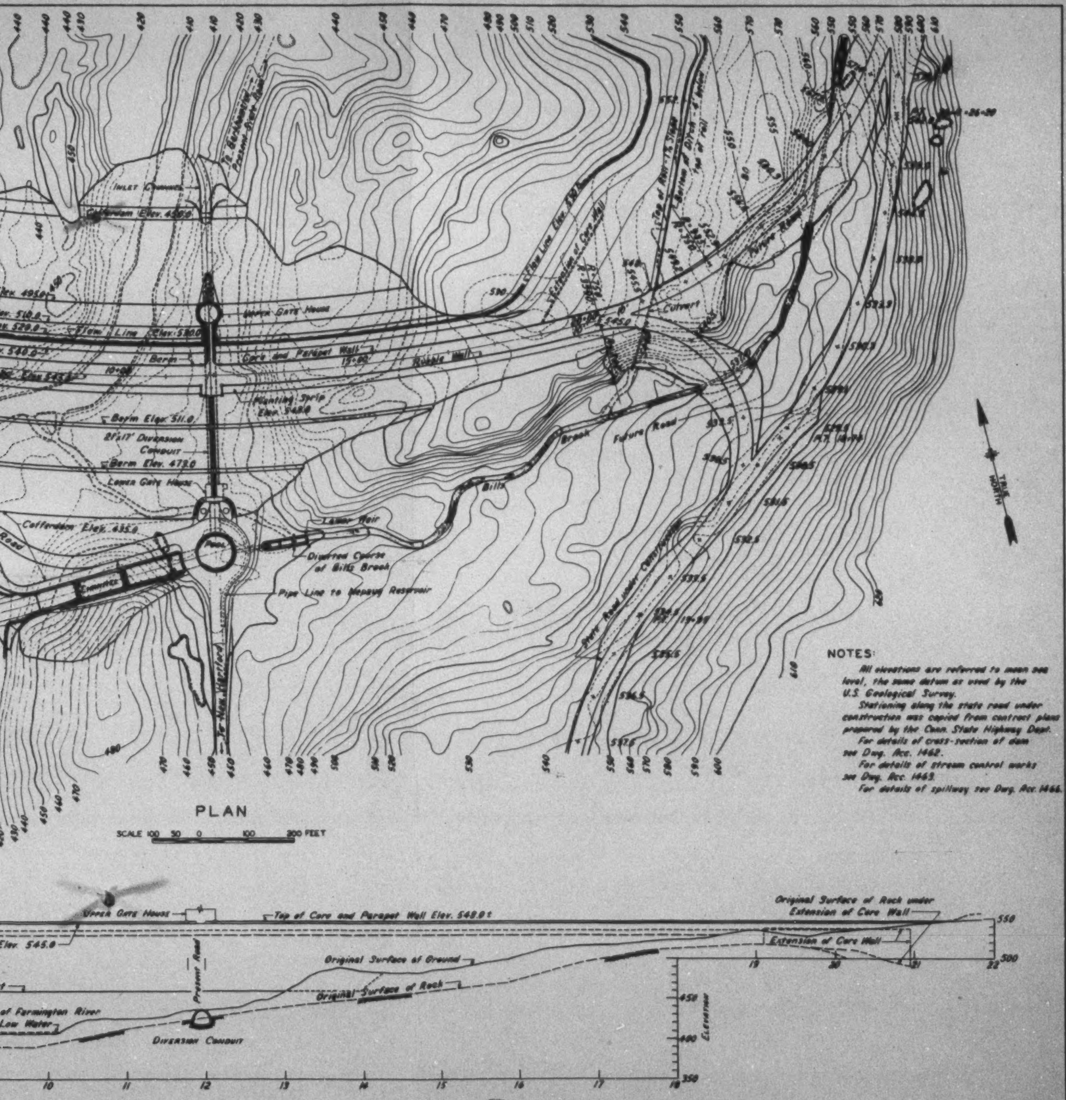
PLANS AND SPECIFICATIONS APPROVED, JUNE 16, 1933.

William G. Smith

MEMBER CONNECTICUT STATE BOARD OF CIVIL ENGINEERS

DRAWN W.J.H. & H.C.H.
TRACED H.A.S.
CHECKED K. K.

GENERAL


Inlet Channel
Cofferdam Elev. 485.0
To Barkhamsted Present State Road
Upper Gate House
Flow Line Elev. 530.0
Core and Parapet Wall
Berm
Rubble Wall
Planting Strip Elev. 545.0
Berm Elev. 511.0
21'x17' Diversion Conduit
Berm Elev. 472.0
Lower Gate House
Cofferdam Elev. 435.0
Lower Weir
Diverted Course of Bills Brook
Pipe Line to Nepaug Reservoir
Pool
Spillway
To New Hartford
Bills Brook
Future Road
Culvert
Extension of Core Wall
Top of Rock 1½ Slope
Bottom of Ditch
State Road under Construction
TRUE NORTH
PLAN
SCALE 100 50 0 100 200 FEET
Upper Gate House
Top of Core and Parapet Wall Elev. 548.0±
Elev. 545.0
Present Road
Original Surface of Ground
Original Surface of Rock
of Farmington River
Low Water
Diversion Conduit
Original Surface of Rock under Extension of Core Wall
Extension of Core Wall
ELEVATION
550
500
450
400
350
10
11
12
13
14
15
16
17
18
19
20
21
22
CENTER LINE OF ROAD
120 FEET

NOTES:

All elevations are referred to mean sea level, the same datum as used by the U.S. Geological Survey.

Stationing along the state road under construction was copied from contract plans prepared by the Conn. State Highway Dept.

For details of cross-section of dam see Dwg. Acc. 1462.

For details of stream control works see Dwg. Acc. 1463.

For details of spillway see Dwg. Acc. 1466.

APPROVED JUNE 14, 1933

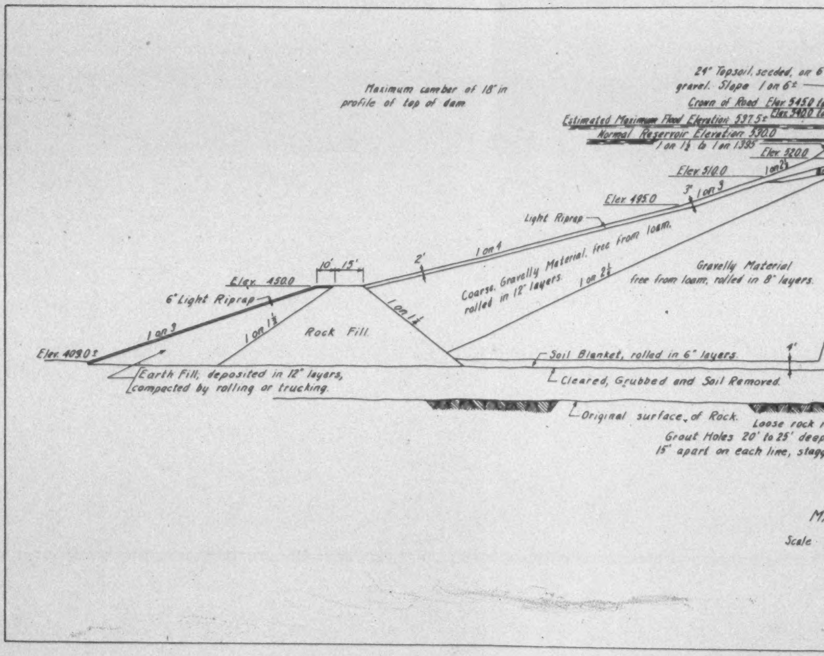
MANAGER AND CHIEF ENGINEER

B. Ridgway

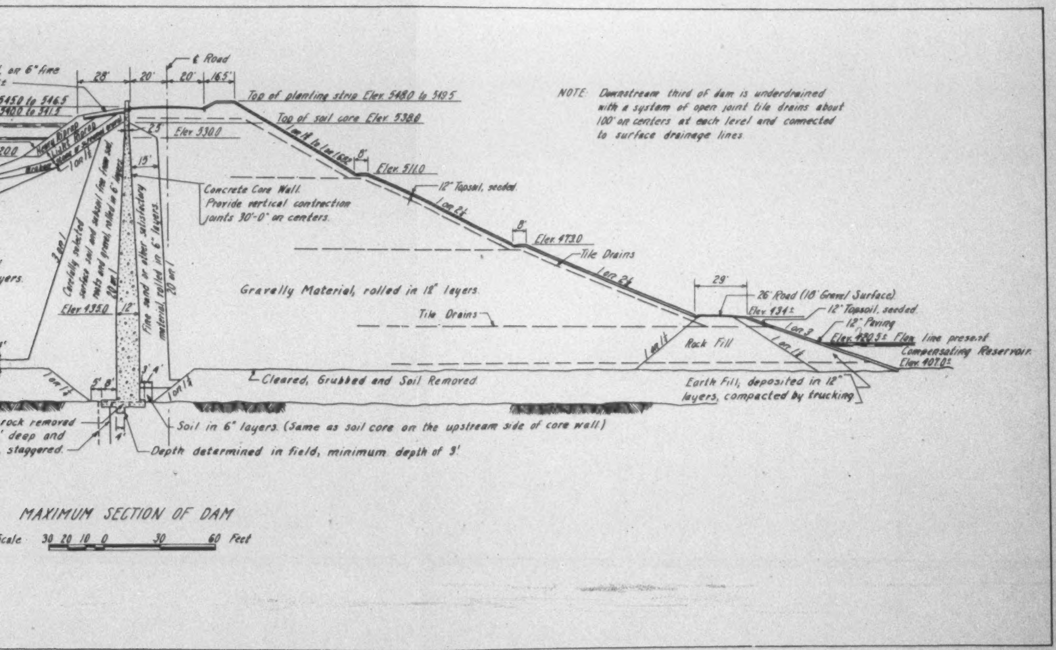
CONSULTING ENGINEERS

AL PLAN OF BILLS BROOK DAM.

PLATE I.

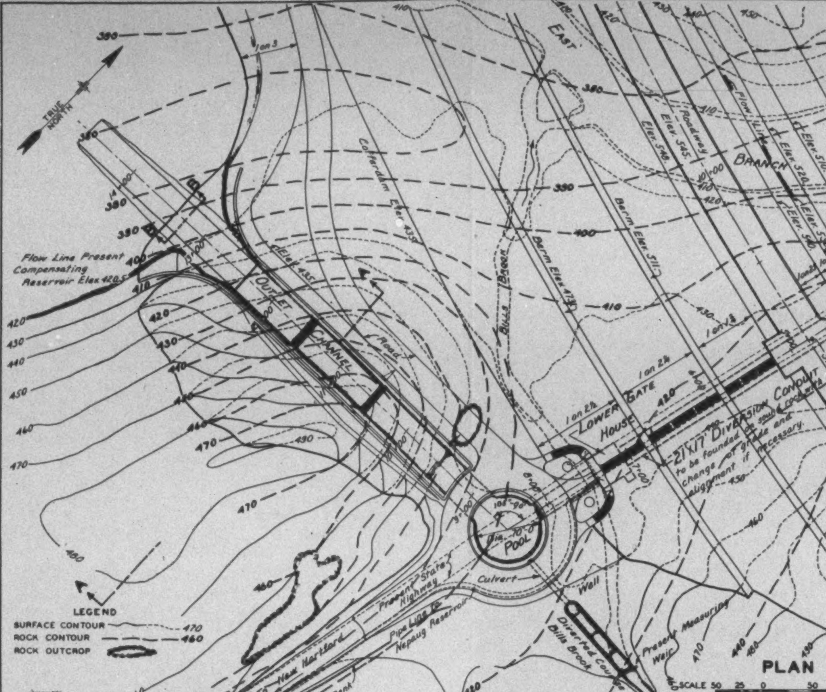

Maximum camber of 18" in profile of top of dam
24" Topsoil, seeded, on 6
gravel. Slope 1 on 6±
Crown of Road Elev. 545.0
Elev. 540.0
Estimated Maximum Flood Elevation 537.5±
Normal Reservoir Elevation 530.0
1 on 1½ to 1 on 1.395
Elev. 520.0
1 on 2½
Elev. 510.0
3'
1 on 3
Elev. 495.0
Light Riprap
1 on 4
2'
10'
15'
Elev. 450.0
6" Light Riprap
1 on 3
1 on 1½
Rock Fill
1 on 1½
Coarse, Gravelly Material, free from loam, rolled in 12" layers.
1 on 2½
Gravelly Material free from loam, rolled in 8" layers.
Elev. 409.0±
Earth Fill; deposited in 12" layers, compacted by rolling or trucking.
Soil Blanket, rolled in 6" layers.
4'
Cleared, Grubbed and Soil Removed.
Original surface of Rock.
Loose rock
Grout Holes 20' to 25' deep
15' apart on each line, stagg
M
Scale

CROSS-SEC

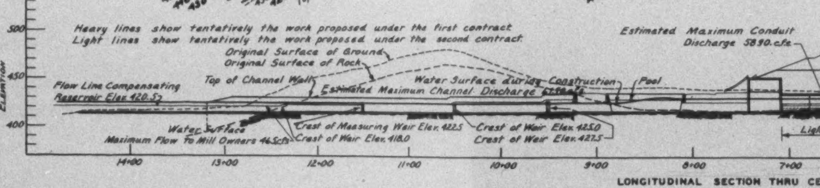

, on 6" fine
t
5450 to 546.5
5400 to 541.5
200
Heavy Riprap
Light Riprap
Broken stone or screened gravel
1 on 1½
28'
20'
20'
16.5'
℄ Road
Top of planting strip Elev. 548.0 to 519.5
Top of soil core Elev. 538.0
2.5'
Elev. 530.0
15'
1 on 1½ to 1 on 1.633
8'
Elev. 511.0
Concrete Core Wall.
Provide vertical contraction
joints 30'-0" on centers.
12" Topsoil, seeded.
1 on 2½
8'
Elev. 473.0
Tile Drains
1 on 2½
3 on 1
Carefully selected surface soil and subsoil free from sod, roots and gravel, rolled in 6" layers.
20 on 1
Fine sand or other satisfactory material, rolled in 6" layers.
20 on 1
yers.
Elev. 435.0
12'
Gravelly Material, rolled in 12" layers.
Tile Drains
29'
26' Road (18' Gravel Surface)
Elev. 434±
12" Topsoil, seeded.
12" Paving
1 on 3
Elev. 420.5± Flow line present
Compensating Reservoir.
Elev. 407.0±
1 on 1½
Rock Fill
1 on 1½
1 on 1½
5'
8'
3'
4'
1 on 1½
Cleared, Grubbed and Soil Removed.
Earth Fill, deposited in 12" layers, compacted by trucking.
rock removed
' deep and
, staggered.
4'
Soil in 6" layers. (Same as soil core on the upstream side of core wall.)
Depth determined in field, minimum depth of 3'.
NOTE: Downstream third of dam is underdrained with a system of open joint tile drains about 100' on centers at each level and connected to surface drainage lines.
MAXIMUM SECTION OF DAM
Scale 30 20 10 0 30 60 Feet

-SECTION OF BILLS BROOK DAM.

PLATE II.



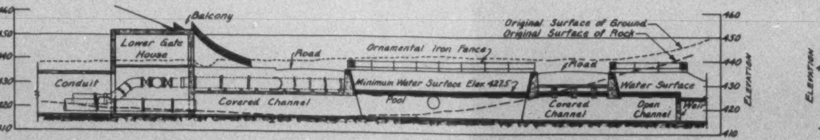
TRUE NORTH
EAST
BRANCH
Flow Line
Roadway Elev. 545
Elev. 548
Berm Elev. 511
Berm Elev. 473
Bills Brook
Cofferdam Elev. 435
Outlet Channel
Road
Flow Line Present Compensating Reservoir Elev. 420.5
Lower Gate House
21'x17' Diversion Conduit to be founded on sound rock with change of grade and alignment if necessary.
Pool
Dia. 70'-0"
Culvert
Well
Present State Highway
Pipe Line to Nepaug Reservoir
To New Hartford
Top of Bank
Diverted Course Bills Brook
Present Measuring Weir
1 on 3
1 on 2½
1 on 1½
LEGEND
SURFACE CONTOUR 470
ROCK CONTOUR 460
ROCK OUTCROP
PLAN
SCALE 50 25 0 50



Heavy lines show tentatively the work proposed under the first contract.
Light lines show tentatively the work proposed under the second contract.
Original Surface of Ground
Original Surface of Rock
Top of Channel Wall
Water Surface during Construction
Pool
Estimated Maximum Channel Discharge
Estimated Maximum Conduit Discharge 5890 c.f.s.
Flow Line Compensating Reservoir Elev. 420.5
Water Surface Maximum Flow to Mill Owners 46 c.f.s.
Crest of Measuring Weir Elev. 422.5
Crest of Weir Elev. 418.0
Crest of Weir Elev. 425.0
Crest of Weir Elev. 427.5
ELEVATION
500
450
400
14+00
13+00
12+00
11+00
10+00
9+00
8+00
7+00

LONGITUDINAL SECTION THRU CE

SCALE 50 25 0 50



Balcony
Lower Gate House
Road
Ornamental Iron Fence
Original Surface of Ground
Original Surface of Rock
Road
Conduit
Covered Channel
Minimum Water Surface Elev. 427.5
Pool
Water Surface
Covered Channel
Open Channel
Weir
ELEVATION
460
450
440
430
420
410

SECTION THRU LOWER GATE HOUSE & POOL

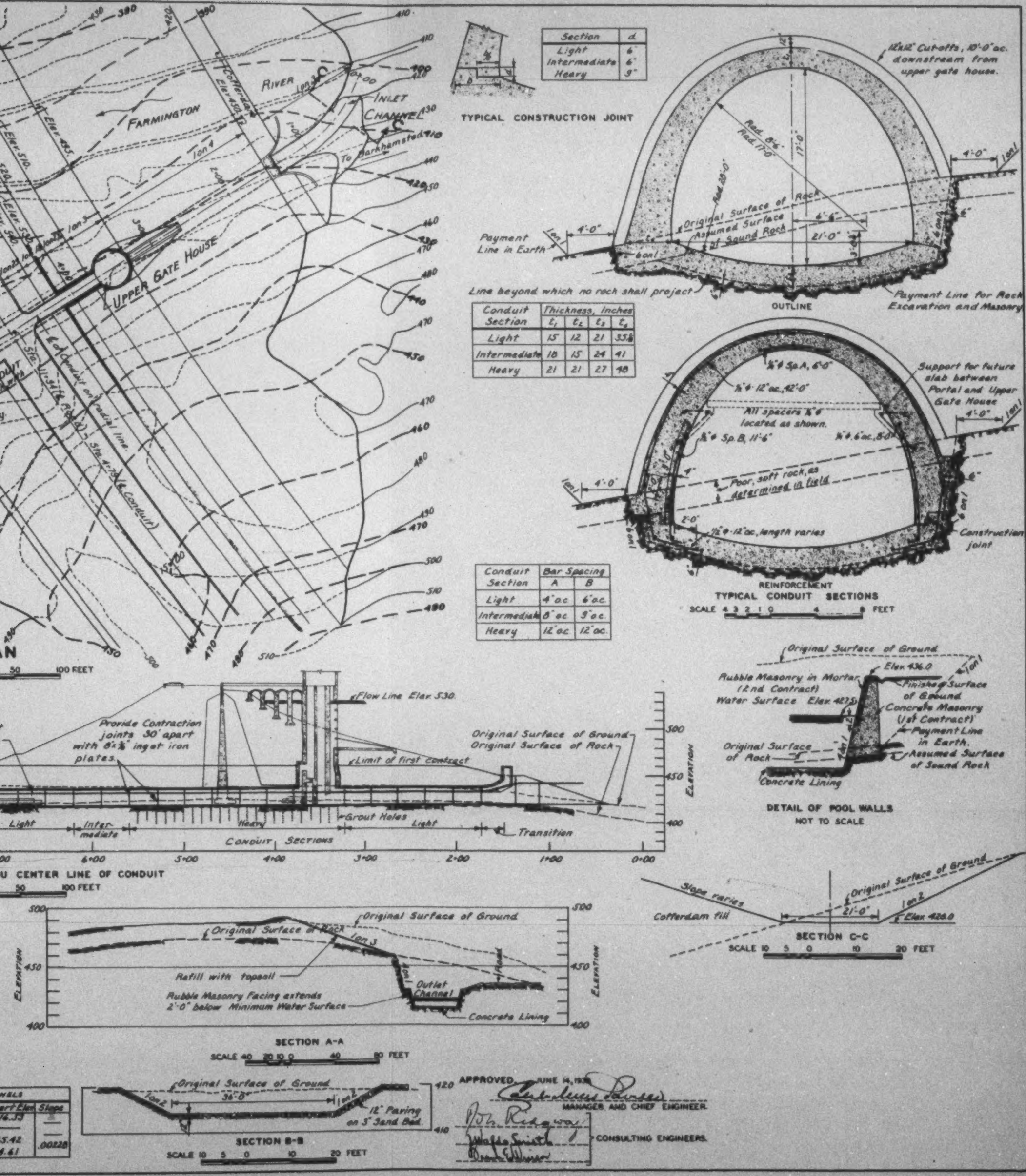
SCALE 20 10 5 0 20 40 FEET

GRADES OF DIVERSION CONDUIT AND CHANNELS							
Sta.	Invert Elev.	Slope	Sta.	Invert Elev.	Slope	Sta.	Invert Ele
0+25	420.00		1+73	420.00	.0065	8+13.75	416.53
1+24	420.00	Level	7+07.25	416.53		Pool	—
1+48	420.00	Level	7+42.25	416.53	Level	8+63.75	415.42
1+73	420.00	Level	8+13.75	416.53	Level	12+37.5	414.61

DRAWN W.J.H. & HAS.
TRACED W.J.H.
CHECKED H.C.H.

PLANS AND SPECIFICATIONS APPROVED, JUNE 16, 1933.
William G. Smith
MEMBER CONNECTICUT STATE BOARD OF CIVIL ENGINEERS.

STREAM CONTR


FARMINGTON
RIVER
INLET CHANNEL
To Barkhamsted
UPPER GATE HOUSE
Cofferdam Elev. 450
Elev. 510
Elev. 485
Conduit on radial line
Sta. 4+70 (℄ Conduit)
15+00
AN
50
100 FEET
TYPICAL CONSTRUCTION JOINT
12x12 Cut-offs, 10'-0" oc. downstream from upper gate house.
Rad. 8'-6"
Rad. 17'-0"
Rad. 28'-0"
17'-0"
6'-6"
21'-0"
4'-0"
1 on 1
6 on 1
Payment Line in Earth
Original Surface of Rock
Assumed Surface of Sound Rock
Line beyond which no rock shall project
Payment Line for Rock Excavation and Masonry
OUTLINE
¾" ∅ Sp. A, 6'-0"
¾" ∅ 12" oc., 12'-0"
All spacers ¾" ∅ located as shown.
¾" ∅ Sp. B, 11'-6"
¾" ∅ 6" oc., 8'-0"
Support for future slab between Portal and Upper Gate House
Poor, soft rock, as determined in field
½" ∅ 12" oc., length varies
Construction joint
REINFORCEMENT
TYPICAL CONDUIT SECTIONS
SCALE 4 3 2 1 0 4 8 FEET
Flow Line Elev. 530.
Provide Contraction joints 30' apart with 8"x⅜" ingot iron plates.
Original Surface of Ground
Original Surface of Rock
Limit of first contract
Light
Intermediate
Heavy
Grout Holes
Transition
CONDUIT SECTIONS
6+00
5+00
4+00
3+00
2+00
1+00
0+00
CENTER LINE OF CONDUIT
50
100 FEET
ELEVATION
500
450
400
Rubble Masonry in Mortar (2nd Contract)
Water Surface Elev. 427.5
Elev. 436.0
Finished Surface of Ground
Concrete Masonry (1st Contract)
Payment Line in Earth.
Assumed Surface of Sound Rock
Concrete Lining
DETAIL OF POOL WALLS
NOT TO SCALE
Slope varies
Cofferdam fill
Elev. 426.0
1 on 2
SECTION C-C
SCALE 10 5 0 10 20 FEET
Refill with topsoil
1 on 3
Rubble Masonry Facing extends 2'-0" below Minimum Water Surface
Outlet Channel
Road
SECTION A-A
SCALE 40 20 10 0 40 80 FEET
NELS
Start Elev.
Slope
16.33
15.42
14.61
.00228
36'-0"
12" Paving on 3" Sand Bed.
420
410
SECTION B-B
SCALE 10 5 0 10 20 FEET

Section	d
Light	6"
Intermediate	6"
Heavy	9"

Conduit Section	Thickness, Inches			
	t_1	t_2	t_3	t_4
Light	15	12	21	35½
Intermediate	18	15	24	41
Heavy	21	21	27	48

Conduit Section	Bar Spacing	
	A	B
Light	4" o.c	6" o.c.
Intermediate	8" o.c	9" o.c.
Heavy	12" o.c	12" o.c.

APPROVED JUNE 14, 193

Caleb Mills Saville
MANAGER AND CHIEF ENGINEER.

Robt. Ridgway
J Waldo Smith
Frank E Winsor
CONSULTING ENGINEERS.

TROL WORKS OF BILLS BROOK DAM.

PLATE III.

downstream cut are refilled with yellow loam. The upstream face of the core wall is backed up with a soil core of yellow loam, 13 ft. thick at the flow line and 53 ft. thick at the base of dam or top of trench, rolled in 6-in. layers. Between this soil core and the upstream face of the dam extends a heavy fill of gravelly material, free from loam, rolled in 8-in. layers covered with a thick blanket of coarse free-draining gravelly material, rolled in 12-in. layers, the outer face of which is protected with riprap varying from 2 to 10 ft. thick. The downstream face of the core wall is backed up with a heavily compacted sand wall 15 ft. thick, rolled in 6-in. layers. Between this sand backing and the downstream face of the dam extends a pervious fill of gravelly material rolled in 12-in. layers, well-provided with a system of drainage lines of open-jointed tile. The downstream face is covered with a 12-in. layer of top soil, mostly black loam, seeded and grassed, and later to be suitably landscaped with shrubbery, etc. The top of the dam is laid out for a state highway 40 ft. curb to curb, with a parapet wall of stone masonry, laid in mortar, along the north side, setting on top of the concrete core wall. On the upstream side of this parapet, there is a 25-ft. planting strip depressed slightly below the road grade, and the south curb is surcharged with a 3-ft. bank of earth carrying a 4-ft. flag walk with 5-ft. planting strip between walk and sloping downstream face of dam. The top of dam is cambered 18 in. to provide for possible settlement of the earth embankment and also to improve the architectural appearance of the structure. The entire area under the dam is cleared, grubbed and top soil removed before placing the fill. The principal quantities involved are approximately 1 500 000 cu. yds of earth and rock fill, 44 000 cu. yd. of concrete in the core wall and 50 000 cu. yd. of riprap.

Stream Control Works. The question of handling the stream flows during construction from 51 sq. mi. of drainage area of the East Branch above the dam site received careful consideration. The greatest flood on record since 1912 in this area occurred on November 3, 1927, and amounted to 142 c.f.s. per sq. mi. Although the primary function of a diversion conduit is the protection of the dam construction for a very short period of years at most, it seemed desirable that its capacity should be equal to this greatest flood. The grade of the outlet end was limited by the necessity of avoiding backwater interference with construction from the present Compensating Reservoir, and the intake end was limited by the river bed elevation, so that too much water would not be left unavailable for discharge. Also the location of the existing town road upstream 1½ miles from the dam site limited the elevation of the water in the construction pond at the conduit intake. Considering all these conditions, the design for the stream control works resulted in a horse-shoe-shaped concrete conduit 21 ft. wide (inside) and 17 ft. high (inside) founded throughout its 600-ft. length on sound rock, built with suitable exterior concrete collars as water-stops and passing entirely through the dam section, including the core wall, on the east side of the valley (Plate III). The conduit floor is at El. 420.0 at upper end and

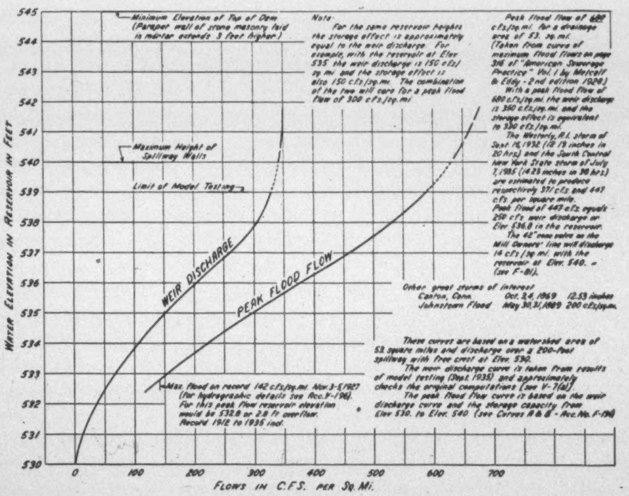
downstream cut are refilled with yellow loam. The upstream face of the core wall is backed up with a soil core of yellow loam, 13 ft. thick at the flow line and 53 ft. thick at the base of dam or top of trench, rolled in 6-in. layers. Between this soil core and the upstream face of the dam extends a heavy fill of gravelly material, free from loam, rolled in 8-in. layers covered with a thick blanket of coarse free-draining gravelly material, rolled in 12-in. layers, the outer face of which is protected with riprap varying from 2 to 10 ft. thick. The downstream face of the core wall is backed up with a heavily compacted sand wall 15 ft. thick, rolled in 6-in. layers. Between this sand backing and the downstream face of the dam extends a pervious fill of gravelly material rolled in 12-in. layers, well-provided with a system of drainage lines of open-jointed tile. The downstream face is covered with a 12-in. layer of top soil, mostly black loam, seeded and grassed, and later to be suitably landscaped with shrubbery, etc. The top of the dam is laid out for a state highway 40 ft. curb to curb, with a parapet wall of stone masonry, laid in mortar, along the north side, setting on top of the concrete core wall. On the upstream side of this parapet, there is a 25-ft. planting strip depressed slightly below the road grade, and the south curb is surcharged with a 3-ft. bank of earth carrying a 4-ft. flag walk with 5-ft. planting strip between walk and sloping downstream face of dam. The top of dam is cambered 18 in. to provide for possible settlement of the earth embankment and also to improve the architectural appearance of the structure. The entire area under the dam is cleared, grubbed and top soil removed before placing the fill. The principal quantities involved are approximately 1 500 000 cu. yds of earth and rock fill, 44 000 cu. yd. of concrete in the core wall and 50 000 cu. yd. of riprap.

Stream Control Works. The question of handling the stream flows during construction from 51 sq. mi. of drainage area of the East Branch above the dam site received careful consideration. The greatest flood on record since 1912 in this area occurred on November 3, 1927, and amounted to 142 c.f.s. per sq. mi. Although the primary function of a diversion conduit is the protection of the dam construction for a very short period of years at most, it seemed desirable that its capacity should be equal to this greatest flood. The grade of the outlet end was limited by the necessity of avoiding backwater interference with construction from the present Compensating Reservoir, and the intake end was limited by the river bed elevation, so that too much water would not be left unavailable for discharge. Also the location of the existing town road upstream 1½ miles from the dam site limited the elevation of the water in the construction pond at the conduit intake. Considering all these conditions, the design for the stream control works resulted in a horse-shoe-shaped concrete conduit 21 ft. wide (inside) and 17 ft. high (inside) founded throughout its 600-ft. length on sound rock, built with suitable exterior concrete collars as water-stops and passing entirely through the dam section, including the core wall, on the east side of the valley (Plate III). The conduit floor is at El. 420.0 at upper end and

El. 416.54 at lower end. This conduit discharges into an open concrete-lined channel 19 ft. wide and about 70 ft. long at the downstream base of the dam, and thence into a 70-ft. concrete pool, which in turn empties into the present Compensating Reservoir via about 500 ft. of concrete-lined channel 36 ft. wide. Both the upper and lower gate houses are located on this conduit so that after the completion of the dam, it may be used as a pipe tunnel to carry two 54-in. pipe lines for water supply to the District and one 54-in. pipe for releasing water to the mill owners for compensation. The recent flood of March, 1936, showed a peak flow of 135 c.f.s. per sq. mi., which served as a great opportunity of checking the hydraulics of the design and also justified the building of a structure of that capacity, although statistical computations indicated the probability of such a flood to be once in 175 years. The design on the basis of 142 c.f.s. per sq. mi. indicated a pond elevation of 444.2 (about 6 ft. below top of upper cofferdam) with maximum flow through the conduit of 115 c.f.s. per sq. mi. Actual observations on the 135-c.f.s. storm showed the maximum pond at El. 443.39 with conduit flow of 106 c.f.s. The difference of 29 c.f.s. per sq. mi. was absorbed in storage in the pond.

Grouting. The rock foundations under both the core wall and diversion conduit were grouted under pressures ranging from 20 to 50 lb. per sq. in. The grout holes under the core wall were arranged in 2 rows, with the holes 15 ft. apart, staggered, and drilled 20 to 25 ft. deep; under the conduit in 3 rows, holes 10 ft. apart and 10 to 15 ft. deep. Additional holes were drilled for testing until absolutely tight conditions in the rock prevailed.

Spillway. Recognizing the importance of an adequate spillway to a reservoir formed by building of an earth dam, very careful consideration was given to the selection of its location, and the problem of its capacity was given much attention and study. The location finally chosen is such as will permit discharge over a rock ridge into a parallel valley, thus removing all danger of washouts. To determine the proper capacity, the hydrological records covering the period from 1912 were studied carefully, as was also the Boston Society of Civil Engineers report of the 1927 flood in New England. The conclusion reached was to provide for a peak flood flow of 500 c.f.s. per sq. mi. from the 53 sq. mi. of watershed with a discharge head at the weir of 7½ ft., so that 7½ ft. of freeboard above the maximum high water would be available. At this point, it might be well to point out that although the freeboard above normal reservoir level is 15 ft. to the road level, there is available in case of emergency, an additional 3 ft. due to the mortared stone parapet wall on the north side of the road and dam. Plotted in graphical form in Figure 1, are the details of the spillway flows and storage effect above the free crest of the weir. On this diagram are noted several interesting comparisons with recent storms of great magnitude and their relation to the spillway capacity. In the case of the remote contingency of a flood far exceeding computations or expectations, it can be readily seen from the curves that by backing up the water to the full height of the channel walls, this spillway will discharge 340 c.f.s. per sq. mi., which aided by the storage of an additional 290 c.f.s. per sq. mi. will take care of a peak flood flow of 630 c.f.s. per sq. mi. with the maximum water still 5 ft. below the road or 8 ft. below the top of the parapet wall. The spillway weir itself is designed with provision for 5-ft. flashboards, and is to be a low concrete weir of ogee section 200 ft. long, curved in plan, and faced with granite masonry. The channel at its upper end between the weir and the stone arch bridge on the road across the dam is all excavated in rock with the cut faced with rubble stone masonry. After passing the divide, the channel continues in a sidehill rock cut 24 ft. wide and in a straight line with the earth cover cut back to a flat slope. The length of this waste channel is about 780 ft. and has a total drop of about 75 ft. Because of the importance of the structure, arrangements were made with Prof. C. M. Allen of the Worcester Polytechnic Institute to conduct a series of model tests of the spillway at the Alden Hydraulic Laboratory. This matter is taken up later in this paper.



Note.
For the same reservoir heights the storage effect is approximately equal to the weir discharge. For example, with the reservoir at Elev. 535 the weir discharge is 150 c.f.s./sq. mi. and the storage effect is also 150 c.f.s./sq.mi. The combination of the two will care for a peak flood flow of 300 c.f.s./sq.mi.

Peak flood flow of 680 c.f.s./sq.mi. for a drainage area of 53. sq.mi. (Taken from curve of maximum flood flows on page 316 of "American Sewerage Practice" Vol. I by Metcalf & Eddy - 2nd edition 1928.)

With a peak flood flow of 680 c.f.s./sq.mi. the weir discharge is 350 c.f.s./sq.mi. and the storage effect is equivalent to 330 c.f.s./sq.mi.

The Westerly, R.I. storm of Sept. 16, 1932 (12.13 inches in 20 hrs.) and the South Central New York State storm of July 7, 1935 (14.23 inches in 30 hrs.) are estimated to produce respectively 371 c.f.s. and 447 c.f.s. per square mile.
Peak flood of 447 c.f.s. equals 250 c.f.s. weir discharge or Elev. 536.8 in the reservoir.

The 42" cone valve on the Mill Owners' line will discharge 14 c.f.s./sq.mi. with the reservoir at Elev. 540. - (see F-81).

Other great storms of interest

Canton, Conn.	Oct. 3, 4, 1869	12.53 inches
Johnstown Flood	May 30, 31, 1889	200 c.f.s./sq.mi.

These curves are based on a watershed area of 53 square miles and discharge over a 200-foot spillway with free crest at Elev. 530.

The weir discharge curve is taken from results of model testing (Sept. 1935) and approximately checks the original computations (see V-7(a)).

The peak flood flow curve is based on the weir discharge curve and the storage capacity from Elev. 530. to Elev. 540 (see Curves A & B - Acc. No. F-196)

Fig. 1. — Spillway Flows and Storage Effect.

Gate Houses and Diversion Conduit. The upper gate house is a circular building about 40 ft. in diameter, founded on sound rock foundation, extending up through the upstream embankment about 100 ft. from the road and connected with it by a 3-span granite arch bridge. This building is to house the main control gates on the various pipe lines, the principal ones to be electrically-operated. The layout includes several wells provided with stop-log guides and screens, etc., arranged so there is double control of operation. The circular superstructure is to be faced with granite ashlar masonry and covered with a conical roof, slate surfaced.

The lower gate house is generally rectangular in shape, about 53 ft. by 33 ft., backed up against the downstream face of the dam with only the front wall and roof exposed. The exposed portions are to be finished in stone masonry with a straight stone stairway up the dam to the top road and semi-circular stone stairways at each end to the lower road. Below this building are the pool and channel comprising the outlet works, now in concrete, but also to be faced with stone masonry for architectural appearance as well as for protection of the concrete against weathering. The lower gate house will contain some additional gates on the various pipe lines and cross-connections, also the Venturi meters and power plant.

The diversion conduit will contain the pipe lines extending between these buildings, and from the lower chamber one line will be extended down the road past the pool for discharging water to Nepaug Reservoir. The mill owners' line will discharge into the basement under the sub-floor of the lower gate house. The design of the discharge end of this line was complicated with high-velocity flows and the question of the dissipation of of considerable energy. After considerable study of this problem, model testing was resorted to, details of which are explained further on.

Miscellaneous Features. The work includes also a service road, the permanent diversion of Bills Brook and an extensive landscaping program.

Entering from the main state highway about 3/8 mile south of the lower gate house, a 30-ft. road extends to the building, circles the pool and extends across the lower cofferdam following alongside the outlet channel, as a 26-ft. road, crosses the upper end of the Compensating Reservoir, follows down the west bank for less than 1/4 mile to a 200-ft. circular parking station, then extends in a generally northerly direction for about 1/2 mile to the main state highway just beyond the west end of the dam, crossing the spillway channel on a stone arch bridge about 450 ft. downstream from the dam. This road has been so located as to serve as a temporary road in case of floods during completion of the dam, and as a permanent service road afterwards for ease in operation and maintenance, as well as a scenic driveway.

The permanent diversion of Bills Brook, a watershed of 1 3/4 sq. mi., is to be by means of a gate-controlled channel through a natural gulley just northeast of the dam, arranged so that the flows diverted into the reservoir can be regulated, allowing some water to discharge down the present channel over a weir into a stone masonry well alongside the circular road at the lower gate house.

It has always been felt that such an important work as the dam and reservoir should not only be safe and practicable, but that, since it was being built for all times, it should also respond to architectural treatment, both in structures and in landscape. Consequently, it is planned to landscape appropriately both the dam and the adjoining areas.

When completed, the Bills Brook Dam will impound about 30 billion gallons of water, forming the Barkhamsted Reservoir about 8½ miles long extending to less than ½ mile from the Massachusetts state line. The maximum width of the reservoir will be about ¾ of a mile and the average depth of water about 40 ft. The area to be flooded is about 2 350 acres with a shore line of about 31 miles.

Model Testing.

Spillway. The object of the spillway model testing was to study the hydraulic features of the weir and channel and the matter of erosion at the discharge end where the rock runs out below the channel grades. The model was built to a scale of one-thirtieth full size and followed exactly the preliminary design drawings. Some portions of the model were built of wood and others of sawdust concrete. After calibrating the measuring weir in the laboratory by discharge into a large weighing tank, the first test was the calibration of the spillway weir itself to determine the relation between discharge and water level in the reservoir. A series of tests was then run to determine the water-surface profiles along both sides of the channel, and finally the probable limits of erosion at the discharge end of channel were studied. Observations made during the testing suggested certain minor changes in the floor grades which were made, the erosion tests indicated the best results with the outlet or discharge end changed in shape to a level floor for the end 50 ft. and the walls flared in this section to give an end width of double the main channel section. After these changes, the tests were rerun. In general, the model testing checked the design computations, indicated sufficient safety along the channel with lower walls and at the outlet showed insignificant erosion conditions, free from any erosion under or around the walls. Just beyond the end of the floor and walls riprapping the bottom and sides of the channel will sufficiently protect the adjacent ground against damage. The tests resulted in reducing the construction cost of structure by considerably more than the cost of the model work.

Diversion Conduit. The location of the regulating valve in the lower gate house on the end of the mill owners' pipe is such that it is desirable to discharge the water with as little commotion as possible and so as not to create eddies or waves to disturb the smooth flow desired in the pool outside. This valve is to be located about 110 ft. below the reservoir level and by agreement its maximum discharge must be 300 m.g.d. under all reservoir conditions. Therefore, high velocities are unavoidable, and the computations indicated the possibility of vacuum conditions in the downstream section of the valve particularly when throttled. The object of this model testing, carried on at the Alden Hydraulic Laboratory, was to check the design for capacity of the layout and study the vacuum conditions to determine the proper shape of outlet channel between the valve and pool. The

CONTRACT No. 15.

For the Construction of the Stream Control Works and the Lower Portion of Bills Brook Dam.

C & R Construction Co., Boston, Mass.

Item.	Description.	Unit.	Quantity.	Price.	Amount.
1	Clearing and grubbing	acre	10	$50.00	$500
2	Soil removal	cu. yd.	27 000	.30	8 100
3	Stream control during construction	lump sum	......	.....	1 000
4	Earth excavation	cu. yd.	120 000	.40	48 000
5	Rock excavation	cu. yd.	40 000	1.00	40 000
6	Pumping	lump sum	......	.....	500
7	Special preparation of rock surfaces	sq. yd.	4 000	.50	2 000
8	Earth fill in cofferdams	cu. yd.	25 000	.40	10 000
9	Rock fill in cofferdams	cu. yd.	36 000	.30	10 800
10	Riprap	cu. yd.	2 700	1.00	2 700
11	General fill, rolled in 12-in. layers	cu. yd.	350 000	.30	105 000
12	General fill, rolled in 8-in. layers	cu. yd.	85 000	.30	25 500
13	Soil core and soil blanket	cu. yd.	75 000	.35	26 250
14	Selected fill, rolled in 6-in. layers	cu. yd.	20 000	.35	7 000
15	Topsoil, dressing and seeding	cu. yd.	6 000	.75	4 500
16	Tile drains—8-in. and less in diameter	lin. ft.	2 500	.35	875
17	Tile drains—10-in. to 15-in. in diameter	lin. ft.	3 700	.75	2 775
18	Catch basins	basin	9	70.00	630
19	Paving	cu. yd.	1 500	1.50	2 250
20	Concrete masonry—core wall	cu. yd.	16 500	4.50	74 250
21	Concrete masonry—diversion conduit and portal	cu. yd.	3 200	5.80	18 560
22	Concrete masonry—upper and lower gate houses	cu. yd.	1 800	7.00	12 600
23	Concrete masonry—retaining walls	cu. yd.	3 400	4.90	16 660
24	Concrete masonry—lining in channels	cu. yd.	1 350	5.00	6 750
25	Drillings holes in rock and masonry	lin. ft.	5 000	.20	1 000
26	Furnishing and placing grout	cu. yd.	100	8.00	800
27	Timber structures	m. ft. B.M.	85	60.00	5 100
28	Portland cement	barrel	37 000	2.30	85 100
29	Broken stone, gravel and sand	cu. yd.	100	3.00	300
30	Metal for reinforcing concrete and water-stops	pound	350 000	.0325	11 375
31	Miscellaneous cast iron, wrought iron and steel	pound	16 000	.08	1 280
32	Waterproofing	sq. ft.	9 000	.40	3 600
33	Cleaning up	lump sum	......	.....	3 000
	Total				$538 755

Contract No. 17.

For the Construction of the Second Portion of Bills Brook Dam.

B. Perini & Sons, Inc., Framingham, Mass.

Item.	Description.	Unit.	Quantity.	Price.	Amount.
1	Clearing and grubbing..........	acre	10	$86.00	$860.00
2	Soil removal..................	cu. yd.	12 000	.28	3 360.00
3	Earth excavation...............	cu. yd.	110 000	.18	19 800.00
4	Rock excavation for core wall....	cu. yd.	3 000	4.00	12 000.00
5	Rock excavation (blasting permitted)	cu. yd.	3 000	3.70	11 100.00
6	Special preparation of rock surfaces	sq. yd.	1 500	.58	870.00
7	Rock fill......................	cu. yd.	700	1.00	700.00
8	Riprap.......................	cu. yd.	25 000	1.53	38 250.00
9	General fill, rolled in 12-in. layers	cu. yd.	500 000	.23	115 000.00
10	General fill, rolled in 8-in. layers..	cu. yd.	140 000	.23	32 200.00
11	Soil core and soil blanket........	cu. yd.	75 000	.61	45 750.00
12	Selected fill, rolled in 6-in. layers..	cu. yd.	45 000	.28	12 600.00
13	Topsoil, dressing and seeding.....	cu. yd.	13 000	1.00	13 000.00
14	Tile drains—8-in. and less in diameter........................	lin. ft.	4 300	.50	2 150.00
15	Tile drains—10-in. to 15-in. in diameter....................	lin. ft.	5 500	.75	4 125.00
16	Catch basins..................	basin	48	75.00	3 600.00
17	Paving.......................	cu. yd.	850	4.50	3 825.00
18	Concrete posts................	post	142	3.00	426.00
19	Concrete masonry—core wall.....	cu. yd.	21 000	4.35	91 350.00
20	Concrete masonry — upper gate house......................	cu. yd.	1 100	9.20	10 120.00
21	Granite facing—upper gate house	sq. ft.	3 000	4.70	14 100.00
22	Granite curbing...............	lin. ft.	1 150	3.55	4 082.50
23	Drilling holes in rock and masonry	lin. ft.	5 000	.40	2 000.00
24	Furnishing and placing grout.....	cu. yd.	100	17.00	1 700.00
25	Portland cement...............	barrel	28 000	2.75	77 000.00
26	Broken stone or gravel	cu. yd.	8 500	1.20	10 200.00
27	Wooden bridge and railings......	lump sum	......	.....	1 500.00
28	Gravel surfacing...............	cu. yd.	1 600	2.00	3 200.00
29	Metal for reinforcing concrete and water-stops.................	pound	230 000	.041	9 430.00
30	Miscellaneous cast iron, wrought iron and steel...............	pound	26 000	.12	3 120.00
31	Wire rope railing..............	lin. ft.	2 700	.45	1 215.00
32	Anchorages...................	anchorage	12	10.00	120.00
33	Waterproofing................	sq. ft.	3 300	.25	825.00
34	Cleaning up..................	lump sum	......	.....	1 200.00
	Total.....................				$550 778.50

model was built of wood to a scale of one-sixteenth full size and certain portions glassed, so that the flows could be observed. In general, the results check the design, and by testing various shaped outlets under all operating conditions, a layout was evolved in which the valve can be operated with positive pressure conditions for all ranges of reservoir height to discharge flows within the required limits; that is, the maximum of 300 m.g.d. For extremely high reservoir condition, there is a small range of valve opening just under a wide open valve, which produces a maximum vacuum of 5 lb. per sq. in. This position of the valve discharges in the neighborhood of 400 m.g.d., very much in excess of expected operating conditions, so that this small area of vacuum possibility is not considered serious, since the valve will probably never be operated that way. The surface of the water discharged into the pool is smooth and calm, varying in all about 4 in. in elevation throughout the entire area of about 5 000 sq. ft. The cost of the model testing is justified by the single fact that a layout has resulted in which the immense amount of energy is dissipated without causing a lot of commotion and disturbance, and in which no vacuum occurs within the normal operating range of the valve, thus insuring the permanency of the structure and preserving the architectural features unspoiled by disturbed flows of water.

Miscellaneous.

Costs. The contract prices for Contracts 15 and 17, covering the construction of Bills Brook Dam as far as construction has progressed are summarized in the following two schedules. Bids received varied in totals from $538 755 to $895 280 for Contract 15 and from $550 778.50 to $759 525 for Contract 17.

Engineers. The general design drawings were approved by Mr. Caleb Mills Saville, Manager and Chief Engineer, by a Board of Consulting Engineers, consisting of the late J. Waldo Smith, formerly Chief Engineer, Board of Water Supply of New York City, Mr. Robert Ridgway, a member of the Hoover Dam Commission, and Mr. Frank E. Winsor, Chief Engineer, Metropolitan Water Supply Commission, Boston, Mass., and by Mr. William G. Smith of Waterbury, Member of the Connecticut State Board of Civil Engineers, who has given general supervision for the State.

Consulting advice on architectural and landscape matters has been received as the work progressed from Mr. Arthur A. Shurcliff, Landscape Architect, Boston, Mass. Structural design of the more important details has been approved by Mr. J. R. Worcester, Consulting Engineer, Boston, Mass.

NEW WATER SUPPLY SYSTEM AND FILTER PLANT AT HINSDALE, N. H.

BY ARTHUR L. SHAW.*

[*Read December 19, 1935*]

Among the New England towns which took advantage of the first Federal Public Works Administration grants, under the provisions of Title II of the National Industrial Recovery Act of 1933, was Hinsdale, N. H. The fifteen hundred people who live and work in the central and more closely built part of Hinsdale depended upon springs and wells for their drinking water, and many found it necessary to carry water during protracted dry periods each year. For fire protection, they relied upon the Ashuelot River which flows through the village but which is unsuitable for drinking, because of pollution by sewage and industrial wastes from upstream towns and industries. Some of the mills lying along the river have fire pumps to serve their sprinkler systems, and certain of these pumps were connected with a cast-iron main laid in 1910 along the principal street of the village and having branches up the hillside and across the river into parts of the residential sections. In case of fire, word was sent to the mills, a pump was started, and in course of time, after the partly empty pipe had been filled, pressure became available. Aside from this supply, dependence had to be placed upon fire pumpers drawing directly from the river or from small wells in the outlying sections. In view of the obvious limitations of these facilities, the town authorities decided to undertake the construction of a new water supply system suitable for the usual domestic purposes as well as adequate for fire protection, taking advantage of the opportunity to obtain Federal aid.

Source of Supply. It was hoped that a suitable ground water supply could be found. The inhabitants of Hinsdale have long been accustomed to the colorless waters of wells and springs, and would be critical of the color known to be present in the small streams most likely to be found available for a surface water supply. A thorough reconnaissance, however, including the driving of a total of about 400 ft. of test wells in various localities, at an expense of about $900, failed to discover an area where an adequate supply could be obtained by driven wells. In those localities where there was sufficient overburden to permit driven wells, the fine silty sand of the Connecticut River valley prevailed and yielded little or no water. Drilled wells were not considered feasible, and attention was therefore turned to surface water sources.

After careful consideration of several of the small streams in the vicinity of Hinsdale, it was finally decided to utilize Kilburn Brook which rises in the high hills north of the Village (Fig. 1) and which could be readily diverted to serve the village by gravity at a point about 8 000 ft. distant from the village and about 350 ft. above the river. Above this point, the brook flows through wooded and uninhabited territory. The drainage area above the point of diversion is about 2.71 sq. mi., and Kilburn Pond, a natural body of water covering about 21 acres, with a drainage area of about 1.63 sq. mi. and an elevation of about 470 ft. above the point of diversion, lent itself readily to devėlopment as an impounding reservoir.

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*Of Metcalf & Eddy, Engineers, Boston, Mass.

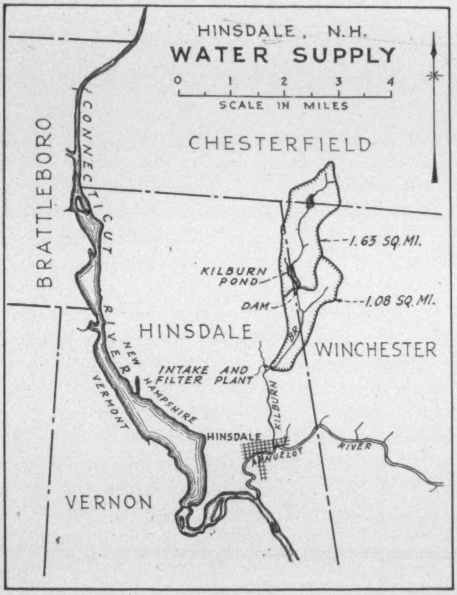
HINSDALE, N.H.
WATER SUPPLY
0 1 2 3 4
SCALE IN MILES
CHESTERFIELD
BRATTLEBORO
CONNECTICUT RIVER
1.63 SQ.MI.
KILBURN POND
DAM
1.08 SQ.MI.
HINSDALE
BR.
WINCHESTER
NEW HAMPSHIRE
VERMONT
INTAKE AND FILTER PLANT
KILBURN
HINSDALE
ASHUELOT RIVER
VERNON

Fig. 1.

Impounding Dam. At the outlet of Kilburn Pond a narrow rock gorge containing the remains of an ancient timber dam was utilized as a site for a new concrete dam, with spillway at El. 1040 or about 7.5 ft. above the natural level of the pond, increasing the flooded area to about 35 acres. Water from this reservoir will be drawn off and allowed to flow down the natural bed of the brook to the small intake reservoir at the point of diversion. The rate of draft will be regulated by gates as may be required to supplement the natural flow from the drainage area below the Kilburn Pond reservoir. About 70 m.g. of water can be stored above the level of the natural surface of Kilburn Pond, providing an estimated safe yield of at least 500 000 g.p.d. at the point of diversion, or several times the initial requirements of the village which are estimated to be from 75 000 to 100 000 g.p.d.

In general, the banks of the reservoir are steep and rocky. As in most natural basins, there is some shallow flowage, mainly at the upper end, but on the whole the reservoir site is an excellent one.

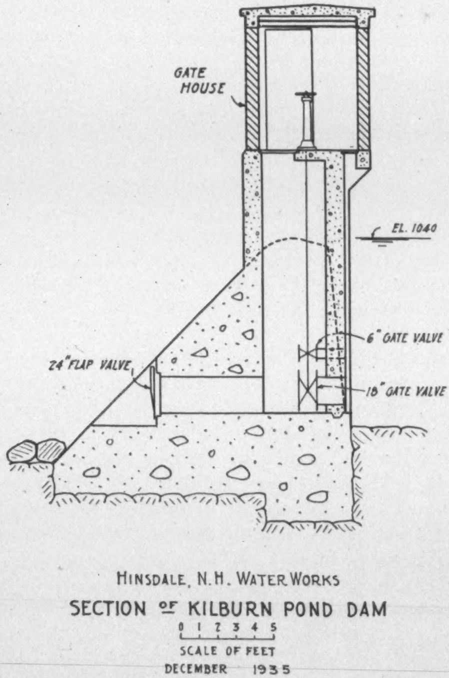
GATE HOUSE
EL. 1040
6" GATE VALVE
24" FLAP VALVE
18" GATE VALVE
HINSDALE, N.H. WATER WORKS
SECTION OF KILBURN POND DAM
0 1 2 3 4 5
SCALE OF FEET
DECEMBER 1935

Fig. 2.

The dam is about 35 ft. in length with a maximum depth to a cut-off in bed rock of about 15 ft. (Fig. 2). It was constructed of cyclopean concrete using fragments of ledge excavated for the footings. At the easterly end is located a small brick gate house inclosing the operating stands of the gates, 6 and 18 in. in diameter, by means of which the flow of water through the dam may be controlled. The spillway section is 27 ft. in length, the effective spillway being reduced to 26 ft. by a small pier in the center, which carries the foot bridge leading to the gate house. The freeboard between the spillway crest and the under side of the foot bridge is 4 ft. and a 3-ft. depth over the spillway will discharge a flood runoff of about 300 c.f.s. per square mile from the drainage area lying above the dam.

It was necessary for the contractor to locate and build a tote road from the highway to the dam site, a distance of approximately 1¼ miles. The dam was built in the dead of winter, and the cold at the altitude of 1 000 ft. above sea level was often intense. All materials for concrete, except cement, were preheated, and the contractor was permitted to use not over 2 lb. of calcium chloride per sack of cement dissolved in the mixing water. Fresh concrete was covered with a tent of tarpaulins and kept from freezing by the use of steam and by wood-burning stoves.

After the snow had left the ground, all trees and brush were removed from the area to be flooded, and all surface débris was raked up and burned. No attempt was made to grub the surface or strip the top soil from the flooded area. The work was completed, and storage of water commenced in time so that the reservoir filled to the crest of the spillway and overflowed in the early summer of 1935.

Intake Dam. The intake dam with its crest at El. 568 is thrown across Kilburn Brook upstream of its highest crossing of Chesterfield Road and just above the location of the purification plant, creating a small reservoir of about 300 000 gal. capacity from which water can be diverted into the purification plant, or in case of need, drawn directly into the pipe line leading to the village. The dam has a spillway section about 45 ft. in length of cyclopean concrete founded on ledge about 13 ft. below the crest. Abutments and wings are of concrete and earth fill. In the north abutment is located the intake well from which the 8-in. cast-iron supply pipe is led through the dam. The well is protected by bar racks with a clear spacing of 1 in. behind which are double removable screens of heavy copper wire with 3 meshes to the inch. A 24-in. waste gate and conduit are also located in the north abutment.

The spillway is designed to pass a flood flow of 300 c.f.s. per square mile from the drainage area above the dam, with a depth of flow of 3 ft. A freeboard of 4 ft. is provided. The spillways of both the intake dam and the Kilburn Pond dam will be adequate for the run-off from rare floods such as that which occurred in parts of New Hampshire and Vermont in 1927.

Purification Plant. The color of the water of Kilburn Brook is high, being often over 100 p.p.m., and the pH often as low as 5.0, which would probably be aggressive toward pipes and services in the distribution system. The forest growth which covers the watershed is largely deciduous, and there is a heavy blanket of leaf mold on the ground which doubtless accounts for the color of the water, and probably for the low pH also. In order to reduce the color and to correct the pH, a mechanical filter plant was constructed just below the intake dam, with a capacity of 200 000 g.p.d. (Fig. 3).

It is expected that for the present and for some time to come, single-shift operation will suffice to provide all the filtered water required by the community. Draft from the standpipe near the village will supply the demand when the filter plant is shut down.

Water flows from the intake reservoir into a welded-steel mixing tank, 7 ft. in diameter, and with a water depth of 11 ft., the water level in which is controlled by a float valve on the inlet. Slow-moving motor-driven wood paddles gently agitate the contents of the tank, and preliminary and more violent mixing is obtained in a baffled spiral channel constructed on the inside of the tank just above the water surface, into which alum and soda ash are fed dry by manually-controlled automatic machines and down which the incoming raw water flows upon entering the tank.

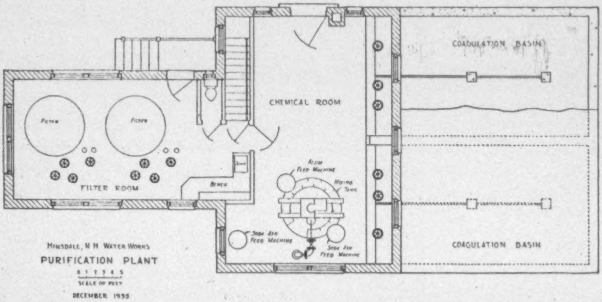
COAGULATION BASIN
CHEMICAL ROOM
Filter
Filter
FILTER ROOM
Bench
Alum Feed Machine
Mixing Tank
Soda Ash Feed Machine
Soda Ash Feed Machine
COAGULATION BASIN
HINSDALE, N H WATER WORKS
PURIFICATION PLANT
0 1 2 3 4 5
SCALE OF FEET
DECEMBER 1935

FIG. 3.

From the bottom of the mixing tank, the treated water passes into two covered coagulation tanks of reinforced concrete, each of which is 14 ft. in width and 25 ft. in length, with an average water depth of 8.5 ft. A timber baffle divides each tank lengthwise into compartments 7 ft. in width. The water enters and leaves the tanks from the same end, passing around the baffles at the far ends of the tanks and being drawn off from the surface at the outlet end over the edges of weir boxes. Inlet, outlet and drainage gates are operated by gate stands mounted on the ends of the tanks, which project inside the building about 4 ft. to give convenient access to the tanks for inspection of the contents and control of the gates.

From the coagulation basins, the water flows into two mechanical filter units composed of cylindrical welded steel tanks 7 ft. in diameter with perforated cast-iron pipe strainer systems.

The water depth over the top of the sand is 5 ft., and the depth from the top of sand to the center line of the strainer pipes is 4 ft. Influent, effluent, wash and drain valves are controlled manually from gate stands on the operating floor. Two 4-in. self-contained rate controllers limit the rate of filtration, and indicating loss-of-head and rate-of-flow gages are provided on the operating floor.

A small motor-driven centrifugal pump connected to the filtered water line delivers water to a 10 000-gal. welded-steel wash-water tank 12 ft. in diameter and 12 ft. high, located on the hillside near the plant, with its bottom 27 ft. above the wash-water troughs. Rate of washing is controlled by an 8-in. self-contained rate controller. A single wash-water trough is provided in each filter, with lip 30-in. above the sand surface.

Just before leaving the plant, and after passing through a meter, the water receives after-treatment with soda ash to establish a desirable pH before entering the distribution system. This chemical is applied by means of a manually-controlled dry-feed machine, from which the chemical is washed into an open riser on the main pipe by a small jet of water drawn from the wash water tank.

No filtered water basin is provided at the plant, the filters discharging directly into the pipe which leads to the village, and thence into the standpipe. The open riser referred to in the previous paragraphs also serves to admit air to the discharge main to relieve a partial vacuum which would otherwise occur at times, owing to the fact that under certain circumstances the treatment plant is above the hydraulic gradient, and it is therefore necessary to avoid the danger of air-binding in the filters.

The filter house is constructed of red brick with heavy wood floor of mill-type construction; it has a wooden roof with asphalt shingles (Fig. 4). Space is provided for the storage of chemicals on the main floor of the building, convenient to the chemical-feed machines. Chemicals are loaded into the hoppers of the chemical-feed machines by hand, only a small amount being required daily to keep the hoppers full. A small bench and sink are provided, together with apparatus for the simple and routine determinations which are necessary to follow the operation of the plant and to regulate the application of chemicals. The building is heated by an oil-fired hot-air furnace. Wash-water and sludge from the settling basins will be discharged directly into the brook below the plant.

Future extension to a total capacity of 300 000 g.p.d. is provided for in the design of the piping so as to permit the addition of one more filter unit by the extension of the filter wing. The mixing tank and the coagulation basins have sufficient capacity to serve 3 units of filters. Since the color and characteristics of the water of Kilburn Brook vary widely with the seasons and are affected by sudden fluctuations due to local rainfall and the like, it is expected that the required rates of application of chemicals will vary from time to time. The plant has not yet been in operation long enough to establish a final routine, but it has been clearly demonstrated that the process can reduce the color of the water to an acceptable degree. At present about 1½ grains per gallon of alum and about 1 grain per gallon of soda ash are added to the raw water and about 0.6 grain per gallon of soda ash to the filtered water. The color has been reduced from 110 to 8 p.p.m. and the final pH brought up to about 7.5 or 8.

Supply Main. An 8-in. cast-iron supply main extends about 8 000 ft. from the filter plant to the village where it enters the distribution system. In case of need, as for example to make the storage in the intake reservoir available to supplement the storage in the standpipe at the village during extraordinary fire draft, or to quickly replenish the standpipe after depletion by a heavy fire demand, raw water from the intake reservoir can be drawn directly into the supply main by manipulation of valves outside the filter plant, so as to bypass the latter.

A pressure-regulating valve is installed on the supply main near the outskirts of the village to serve the double purpose of preventing the overflow of the standpipe and of protecting the village from the excess head of about 100 ft. which exists between the top of the standpipe and the intake works. The irregularities of the topography along the line of the supply main create several summits in the main as it descends from the purification plant to the village; these are equipped with automatic air valves for the relief of air which collects in the summits when admitted at the open riser in the filter plant or otherwise, and which would impair the capacity of the supply pipe if allowed to remain.

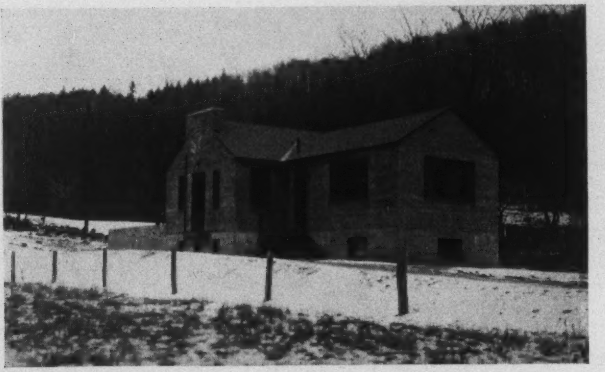

FIG. 4.— HINSDALE, N. H. FILTER PLANT BUILDING.

Distribution System. A covered standpipe 35 ft. in diameter and 35 ft. high, with a capacity of 250 000 gal. was built on the hillside above the village with its top at El. 455. This standpipe serves as a balancing reservoir and as a nearby source of supply to meet fire demands, or to provide for the ordinary requirements of the community while the filter plant is not running.

Cast-iron mains were laid in all the principal streets of the village and connected with the old cast-iron pipe already described in which pressure was previously available only at times of fire. The sizes of the water mains ranged from 6 to 12 in., and the total length of new pipe laid, including the supply main, was about 4 miles (Fig. 5).

Gate valves were provided at all intersections so as to minimize the unprotected area when major repairs are necessary. In certain places along Brattleboro Road where it was desired to avoid cutting a comparatively new concrete pavement, a parallel 2-in. cement-lined wrought-iron pipe was laid to serve the houses on the side of the street opposite that in which the main pipe was located. Elsewhere a single pipe serves each street.

Sand-spun bell-and-spigot pipes hot-dipped in coal-tar varnish were used throughout the distribution system, except as previously noted, and in a few short dead-end streets where 2-in. cement-lined wrought-iron pipes were laid. House services from cast-iron mains consist of ¾-in. brass corporation cocks, lead goose-necks and 1-in. cement-lined wrought-iron service pipes with lead-lined fittings and brass curb cocks. Corporation cocks were omitted on wrought-iron mains. All pipes were laid with 5-ft. cover.

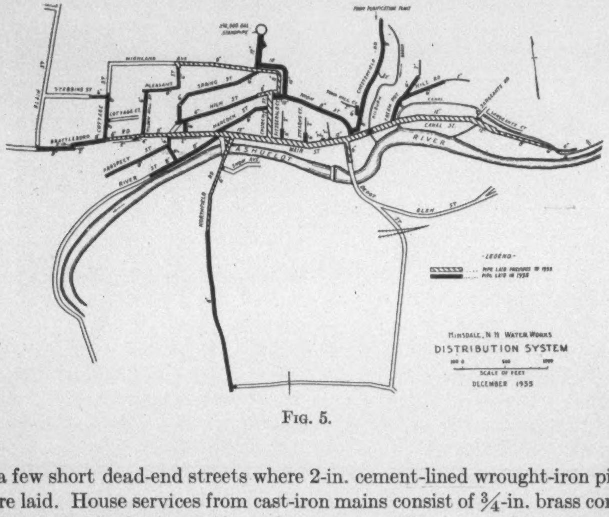
250,000 GAL STANDPIPE
FROM PURIFICATION PLANT
BLAIN ST
STEBBINS ST
COTTAGE ST
COTTAGE CT.
HIGHLAND AVE
PLEASANT ST
SPRING ST
HIGH ST
HADLOCK ST
CHURCH ST
FITZGERALD CT.
STEARNS CT.
BRATTLEBORO RD
PROSPECT ST
RIVER ST
ASHUELOT
RIVER
MAIN ST
NORTHFIELD RD
CHESTERFIELD RD
MILL RD
CANAL
CANAL ST.
SARGEANTS RD
SARGEANTS CT
DEPOT ST
GLEN ST
-LEGEND-
PIPE LAID PREVIOUS TO 1955
PIPE LAID IN 1955
HINSDALE, N H WATER WORKS
DISTRIBUTION SYSTEM
100 0 500 1000
SCALE OF FEET
DECEMBER 1955

FIG. 5.

The distribution system was designed with a view to providing a fire flow everywhere of at least 1 000 g.p.m. with a residual hydrant pressure of from 10 to 20 lb. per sq. in.; and 3 000 g.p.m. with a residual hydrant pressure of about 50 lb. per sq. in. in the high-value district in the center of the village. The static pressure when the standpipe is full is about 100 lb. per sq. in. in the center of the village, and about 40 lb. at the summit of Highland Avenue, the residential street nearest the standpipe.

All new hydrants have two standard hose connections and one pumper connection and are connected to the mains by 6-in. branches with auxiliary gate valves. Hydrants are spaced in general about 500 ft. apart. Practically all of the existing hydrants which were connected with the old main were without auxiliary gates; some of these were replaced by new hydrants, and all were provided with auxiliary gates.

All former connections between the old main and the mill fire-pumps were broken and plugged. The old pipes were found to contain large quantities of sand and gravel which had apparently accumulated from the river water over a period of years. Some of this material was flushed out, using the water wasted from the standpipe after the leakage test, but it was necessary to break into the old mains at several points and to pull through brushes of the type used for sewer cleaning, to dislodge some of the heavier gravel deposits.

Construction Costs. The construction costs of the Hinsdale water supply system were as follows, all work being done by contract under PWA regulations.

Contract No.	Contractor	Subdivision	Cost of Construction
1	O. W. Miller Co.	Kilburn Pond Dam..............	$9 200
2	James A. Munroe.	Intake dam, purification plant and tank foundations...............	34 500
3	Davison Construction Co.	Distribution system..............	74 500
4	Chicago Bridge & Iron Works,	Standpipe and wash-water tank.....	7 900
		Total..........................	$126 100

Contract 1 as has been stated, was built during the winter of 1934–35; the other three contracts were started in the early summer of 1935, and the entire project was practically completed in the fall of 1935.

The project was investigated by a local committee consisting of Prentiss W. Taylor, Chairman, Albert Krumenaker, Ezra B. Pipe, Gustavus Smith and Robert A. Weeks. It was built under the direction of the same committee with the addition of Clarence D. Fay, and with Mr. Krumenaker as chairman. The technical studies and investigations, plans and specifications, and supervision of construction were undertaken by Metcalf & Eddy, Engineers.

The completed system has been turned over to a newly appointed Board of Water and Sewer Commissioners consisting of Robert A. Weeks, Chairman, Harold S. Garfield and Howard I. Streeter, who have appointed C. W. Harris to be superintendent and filter-plant operator.

Many users have already been connected with the system, and others are expected to apply as the advantages of an ample water supply under pressure become appreciated. The new water works were put to a severe practical test when fire broke out in a large mill of the Hinsdale Paper Company in the center of the town, in the early hours of November 9, 1935, only a short time after the standpipe had been filled and the mill fire-pumps disconnected. About a dozen fire streams were used from the new system, under excellent pressure, and with the aid of out-of-town pumpers supplying additional streams of water drawn from the river, the fire was brought under control in about two hours, doubtless preventing a general conflagration in the center of town. The largest of the mill fire-pumps serving the old system was in the building where the fire originated and was inaccessible within a few minutes after the outbreak, so that but little water would have been available under the old régime. In the opinion of many citizens, this single demonstration justified the expense of the new system. It is not often that a new water supply system is subjected to a maximum demand so soon after inauguration, and this experience demonstrates the wisdom of liberality in design even in small communities.

FILTRATION PLANT OF THE MYSTIC VALLEY WATER COMPANY.

BY WALTER H. CHASE* AND T. B. RAY.†

[*Read November 21, 1935.*]

The Mystic Valley Water Company was organized in 1887 and under special charter was allowed to purchase and take water from Palmer and Dean ponds. These ponds were formed in the days of grist mills by damming Coop's Brook, which is fed by approximately 6 sq. mi. of drainage area, including several large springy swamps. The brook and its feeders run almost entirely through low land which is thickly covered by black alder. The surrounding drainage area consists of hard woodland and a few small farms. This area is typical of our section of Connecticut, and all surface water within several miles has the same general objectionable characteristics: high color, swampy odor and taste.

Apparently forty-eight years ago the physical qualities of the water were not questioned, and as the majority of the residents of today were brought up on "Mistuxet Water," as it is commonly called, there was very little agitation for better quality. However, two industrial plants, velvet manufacturers, which had located in Mystic and Stonington partly on account of the soft water supply, began objecting to the high color when light shades of velvet became the vogue several years ago. It was impossible for them to get some of their colors true while using water with a color of 80 to 100 p.p.m.

The velvet mills installed pressure filters, but their filtration costs were high, and during rush seasons they experienced much difficulty in getting desired results as the filters were often run over capacity and chemicals were forced through. This, of course, caused a floc to form in the dye vats. You can imagine the result: a roll of cream velvet with miscellaneous spots of vegetable coloring. The water company, of course, was entirely at fault! But as the velvet companies consumed over 50 per cent. of the water pumped, their complaints were naturally given much consideration. We also received complaints regarding water quality from out-of-town visitors, and it has been claimed, by real estate agents, that prospective customers lost all interest after taking a drink of water.

The color of the raw water varied from 35 p.p.m. during the winter months to 100 p.p.m. during August, with much higher color after either prolonged drought or heavy rains. The taste and odor varied in about the same proportion. However, as the control of the Company was in local hands, no action toward improving the quality was taken.

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* General Manager, Mystic Water Company, Mystic, Conn.
† Division Engineer, Northeastern Water and Electric Service Corporation, Millbury, Mass.

In 1928, a majority of the stock was acquired by a holding company, and it was realized that something must be done to satisfy the velvet companies as they were, besides being the principal consumers, the employers of a large percentage of the residents of the towns, and, therefore, of vital importance to the Water Company.

Preliminary plans were drawn and methods of financing studied, but before construction could be started financial difficulties developed and the controlling interests of the Water Company were transferred to another holding company. This situation was repeated several times, until during the latter part of 1934 a definite promise was made by the present operators, the Northeastern Water and Electric Service Corporation, that a filter plant would be constructed and placed in operation on or before June 1,1935.

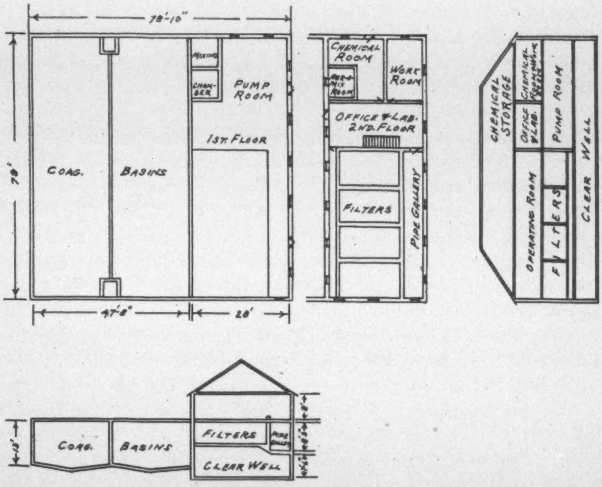
This at last gave us something to work on. The engineering department promptly started revising the various plans which had been drawn in the past in an effort to adopt the better features of all. One item which demanded much attention under present economic conditions was cost. Our aim was to erect a plant for as little money as possible, but to still have a plant that would give efficient and economical operating results. With that point in view we arrived at the plans used for construction. First class materials, workmanship and equipment were specified, all gingerbread was dispensed with, and our plant, to my mind, is a good example of what can be accomplished. Every piece of equipment is doing its intended part, and the plain solid appearance of the building does not clash with the surroundings.

At the time the promise of completion on or before June 1, 1935, was given, it was thought that construction would start before cold weather and that a considerable part of the outside construction would be complete before freezing. This, however, was not accomplished, and we did not break ground until March 21, 1935. The equipment contractor, the Norwood Engineering Company of Florence, Mass., and the general contractors, Elliott and Watrous of Mystic, Conn., agreed that although the time was short, by working together, the job could be completed on time. They then proceeded to demonstrate that two contractors could work together in peace and harmony.

The plans adopted were for a plant of conventional design with a capacity of 2 m.g.d. Serious consideration was given to the steel circular plant designed by Mr. Morse for Hyattsville, Md., and constructed by the Chicago Bridge and Iron Company. There were many interesting features in this compact type of plant. Our objection to it was the possible high cost of maintenance. While this item was claimed to be slight by the sponsors, we did not feel that the data available were sufficient for us to gamble on. We were obliged to decide on a plant that would give definite results at predetermined costs.

The outside dimensions of the plant which were decided on are 79 ft. by 78 ft. 10 in. The mixing chambers, coagulating basins, filters and clear well are of reinforced concrete. The superstructure is of brick with asbestos-shingled roof.

As our time for completion was short, the general contractor was obliged to make numerous sacrifices in his plans of procedure in order to allow the equipment contractor ample time to install the equipment. As the clear well and filters required much forming, the normal procedure would have been to have formed and poured the coagulating basins, this would have allowed the second use of the form lumber and saved the contractor considerable money. However, the sacrifice was made and work started on the clear well. Excavation was started on March 21, erection of the concrete tower on March 26, and on March 30, the footings of the clear well were poured.


78'-10"
78'
47'-8"
28'
Mixing
Chamber
Pump Room
1st Floor
Coag.
Basins
Chemical Room
Work Room
Mix Room
Office & Lab. 2nd Floor
Filters
Pipe Gallery
Chemical Storage
Office & Lab.
Pump Room
Operating Room
Filters
Clear Well
Coag.
Basins
Filters
Pipe Gallery
Clear Well

Filter Plant, Mystic Valley Water Co., Mystic, Conn.

Plant capacity, 2 000 000 gal. per 24 hr.; clear well capacity 124 000 gal.; number of beds, 4; equipment contractor, Norwood Eng. Co.; general contractor, Elliott & Watrous; construction started March 21, 1935, completed May 31, 1935.

Extra precautions were taken to insure a watertight job. Clean crushed granite and bank-run sand were used as aggregates. Celite was employed as an integral water proofing, and the water-cement ratio was kept as low as possible. The bank-run sand was used in preference to washed sand, as it was extremely well graded with only a slight trace of loam. Construction joints were also cut to a minimum. Wherever a construction joint was necessary, a copper diaphragm, extending 8 in. into the concrete on both sides of the joint, was installed.

As the clear well occupies the entire space under the filters, pipe gallery and pump room, it was necessary to complete that and the filters before any equipment could be placed. The bottom slab of the clear well was poured on April 2; on April 4, forms for the clear well walls and top slab were started; several days of rain and snow delayed the form work; but on April 15, the walls and slab were poured. By making one job of pouring, several construction joints were eliminated.

As the back wall of the clear well ties into the bottom slab of the coagulating basin and forms a part of one wall of the coagulating basin, a copper diaphragm was placed at the joint line. Diaphragms were placed in position to insure tight construction joints between the filter walls and the top of the clear well.

A rather complicated set of forms was necessary for this part of the job, as the reinforced columns and beams were at two elevations; also complete destruction of form panels was necessary, due to lack of openings for removal.

Due to more rain and snow, work was again delayed, but on April 22, forms for the filter walls were started. Some progress had also been made on the coagulating basin footings and bottom slab, and on April 23, extra carpenters were put on to form the coagulating basin walls. The filter walls were poured on April 26, and the coagulating basin walls on April 29.

Brick work was started soon after the clear well was poured, and on May 4, the second floor of the superstructure was poured. This gave the equipment contractor space to lay out his work in the pipe gallery. The roof of the coagulating basin was also poured at this time. On May 13, the brick work was complete, and on May 16, the roof was tight. Except for pick-up work, the general contractor had finished his part in 57 calendar days. Considering that the contract was let two days before work was started and that there were several stormy days, very good time was made. There were approximately 900 cu. yd. of excavation and 700 cu. yd. of reinforced concrete. Twelve-inch walls and six-inch slabs were the maximum thicknesses of the concrete.

The equipment contractor had been on the job during the building construction, but other than laying out, setting blow-offs, drains and sleeves, very little had been accomplished. He had, however, been very exact in all of these details which was very important as there were only 48 calendar days remaining for the completion of the job. On May 29, the equipment was, although far from complete, in operating condition. The remaining two days were spent in making adjustments and steadying the plant down in order to produce a stable product at a regular rate. On June 1, at 10.00 A.M. filtered water was turned into the system.

The process of purification consists of aëration, chemical treatment, sedimentation, filtration, pH-correction and chlorination.

The raw water enters the plant by gravity, and is controlled by an hydraulic valve operated by the water level in the coagulating basins, passing thence through a power-operated Aer-O-Mix, for aëration. The soda ash and alum are fed to the Aer-O-Mix by dry-feed machines and are partially mixed during the aëration. The final mix is given in the mixing chamber by a slowly revolving paddle wheel. The water then flows at a low velocity to the inlet fore-bay of the coagulating basins. As the coagulating basin is divided, the fore-bay was necessary for the cutting out of either half of the basin. Activated carbon is added at the inlet fore-bay. The water then flows through diffusing weirs which insure an even distribution over the entire width of the basins and also serve to eliminate any currents through the basin. Four hours are required for the water to pass from the inlet fore-bay to the outlet fore-bay. At the outlet fore-bay the water is collected over a skimming weir, and then flows to a flume which forms one end of the filter beds; influent valves furnish independent control for each bed. The beds are conventional 10 by 18 ft. beds with cast-iron underdrainage system, and the more or less conventional independent air-wash system. Each bed has besides the influent valve, manually operated effluent, wash-water, sewer, rewash, and air valves, also an automatic rate-of-flow control. After passing through the filters the water enters a flume connected with the clear well, where soda ash as a pH-correction and chlorine as a final safety precaution are applied at the entrance to the clear well. The main pump suctions are at the opposite end of the clear well, approximately 75 ft. from the point of final treatment.

The front or operating part of the building is two-story red brick construction with steel sash and green asbestos shingles. Two double doors open on the ground floor, one into the pump room which contains the two main high-lift pumps, Venturi meters, wash-water and air pumps and main heating plant. The other opens into the pipe gallery which extends about half the length of the front of the building and connects with the pump room. This is an ideal location for a pipe gallery as due to its exposed position it cannot be used as a handy place to leave odds and ends. The chlorinator is located in a small room at the extreme end of the pipe gallery.

From the pump room a stairway leads to the office and laboratory on the second floor. The operating floor, chemical room and work room open from the office. The entire operation of the filters is handled from the operating floor by extensions from the various valves. The wash-water and air pumps are also operated by remote-control buttons.

The chemical room contains the Aer-O-Mix, four dry feed machines, the mechanical agitator for the mixing tank and the main switchboard for all low voltage. Chemicals are stored in the loft over the chemical room with provision for charging the dry-feed machines through chutes.

The work room is a very light pleasant room which is equipped with a bench, meter-test table and tools required for the plant.

Another door opens from the laboratory onto the roof of the coagulating basin. This roof was designed with the possibility of nozzle aëration in view, at some future date, if additional aëration became necessary. All of the valves connected with the coagulating basin are operated from the roof.

We have found that best coagulation results with a pH of 5.8 to 5.9. During August, the raw water averaged: color 100 p.p.m., pH 6.2, CO_2 3 p.p.m., alkalinity 3 p.p.m. and hardness 22 p.p.m. By treating with 1.36 grains of alum, 0.43 grain of soda ash per gallon and 7 lb. of Nuchar per million gallons, the results after four hours retention were: color 30 p.p.m., pH 5.8, CO_2 3 p.p.m., alkalinity 3 p.p.m. and hardness 22 p.p.m. The final results after passing through the filter beds and a corrective treatment of 1.08 grains of soda ash per gallon were: color 5 p.p.m., pH 7, CO_2 1 p.p.m., alkalinity 20 p.p.m. and hardness 22 p.p.m.

The final treatment with chlorine is approximately 6.5 lb. per million gallons to obtain a residual of 0.2 p.p.m. after twenty-minute contact.

It has been noted in the coagulating basins that there is a sharp rise in the precipitate shortly after the water leaves the stilling weirs, which holds for about one-third of the length of the basins and then drops off to a small amount at the end of the basins. A small amount of floc is carried over to the beds. Our average filter run under these conditions is 15 hr. with an increase from the initial loss of head of 1.4 to 6.8 ft.

As we do not meter our wash water we are forced to estimate it by checking the amount used against the drawdown in the clear well. We have estimated that 2½ per cent. of the water filtered is used as wash water. To date we have not been bothered by cracks or mud balls in the sand.

THE WATER SYSTEM OF CONCORD, N. H.

BY PERCY R. SANDERS.*

[*Read May 21, 1936.*]

Prior to the building of a water system for Concord, N. H., in 1872, the residents of the city were dependent upon wells, springs and a small aqueduct system. For fire protection, reliance was placed upon large cisterns located throughout the city. As early as 1859, the citizens were impressed with the need for more adequate service, and a committee was formed to investigate all possible sources of supply. After thoroughly going over the situation, they came to the decision that Long Pond now known as Penacook Lake was the best solution of the problem. It is interesting to note in the report of the 1859 committee, that in making an estimate of the amount of water used in Boston the consumption averaged 73 g.p.d. per capita, while in Philadelphia during the year 1852 it was 30.6 ale gallons. The Civil War coming on soon after, no action was taken upon the matter, and the situation remained as it was until 1870, when another committee was formed to go through with the project. This committee made the same recommendations as the 1859 committee. This report was accepted and in May, 1872, work was started on the project.

Penacook Lake is a natural body of water located about three miles northwest of the city and at an elevation 125 ft. higher than the centre of the city at the State House. The lake is 2½ miles long, ½ mile wide and has an area of 368 acres. The shores are rocky, with no swamps, and the watershed rises abruptly from the lake and has an area of 3 sq. mi. The lake is fed mainly by springs, and with a small population on the watershed the natural quality of the water is excellent. The color has an average of 0.5 p.p.m. The estimated yield is 2.6 m.g.d.

An earthwork dam was built 2 000 ft. below the outlet of the original pond and is of the following dimensions: length on bottom 110 ft.; length on top 280 ft.; width on bottom 100 ft.; width on top 20 ft.; perpendicular height 20 ft.; slopes 2 ft. horizontal to 1 ft. vertical. A granite core wall laid in cement was built in the centre, 3½ ft. in thickness at the bottom and 2 ft. at the top. The lake was raised by this dam about 6 ft. A brick conduit 26 by 30 ft., the top of which is 16 ft. below high water, runs from the main part of the lake to the dam.

The right having been acquired to divert 365 m.g. of water per year, it became necessary to adopt some method of measurement. The different kinds of apparatus in use for this purpose were critically examined, resulting in the adoption of Gem meters, manufactured by the National Meter Company of New York. Three 6-in. meters of this patent were attached to the main pipe 200 ft. below the dam. The construction report goes on to state that there are no meters manufactured in the country of greater capacity than these and it is believed to be the only instance where water passing through a main supply pipe is measured by this method. The next year's report gives the registry of these meters as 70 m.g. for six months, less than one half the quantity to which the city is entitled, thus rendering the use of the meters unnecessary for the present and that is the last I can find out about them. I think one of them was later used at the State Hospital.

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*Superintendent, Concord Water Works, Concord, N. H.

The mains, as laid in 1872, were single-thickness wrought-iron, lined and covered with cement. All service pipes were cement-lined. The original main to the city was 14 in. in diameter, and reliance was placed wholly upon this until 1883 when a second cement-lined main 18 in. in diameter was laid. The city was supplied by gravity until 1892 when the growth of the city to the west and into the higher section made some additional means of supply necessary. A pumping station was built in the city and two steam pumps, each with a capacity of 2 m.g.d., were installed. Included in the project was a 2-m.g. reservoir at an elevation 100 ft. higher than the lake. A 20-in. main was also laid through the business section for fire protection. The daily average amount of water pumped at this time was approximately 500 000 g.p.d. Starting in 1897, twenty-five years from the building of the system, a beginning was made on replacing all of the original cement-lined pipe, and this program was continued until all had been replaced.

In 1918, a change was made in the method of pumping from steam to electricity. A rate was given us by the Concord Electric Company of 3⅓ c. per K.W.H. with a maximum yearly payment of $1 800. When our payments to the electric company at this rate amount to $1 800, no further charge is made until we pump 300 m.g. of water. After this we pay $6.00 per m.g. As our pumping averages over 400 m.g. per year, our total cost of pumping is $6.00 per m.g. The power charge at this rate is 1.23 c. per K.W.H. This rate was made with the understanding that all pumping was to be done at night on the hours of the off-peak load, except in emergencies. The first motor-driven pump to be installed was a Worthington 8-in. centrifugal unit, operated by a 100-H.P., A.C. motor with a capacity of 3.8 m.g.d. The second unit was installed in 1926 and is a DeLaval Pump with a capacity of 3.6 m.g.d. These pumps can only be operated singly. The steam pumps and boilers have been kept in condition for emergency use.

In 1926 the Water Board became concerned at the low levels to which the lake periodically fell and employed Metcalf and Eddy to make an investigation as to an additional source of supply. Their recommendations to the Board resulted in the installation of a driven-well system in the Soucook River valley three miles east of the city with all necessary pumping equipment to supply 1.5 m.g.d. to the city. This water is pumped into the high-service system, and this station can be operated singly or in conjunction with the North State Street Station. With these two systems our pumping capacity is 5.3 m.g.d. This plant has been described in a paper read before this Association.*

Construction of dwellings has still continued into the higher sections of the city and has resulted in very low pressures in this area. At present, we are providing for this by the construction of an automatic pumping station with a 250 000-gal. standpipe, the water level of which will be 90 ft. above the reservoir or 190 ft. above the lake. Included in this program is a 250 000-gal. elevated water tower in Penacook, or Ward 1, six miles north of the centre of the city. This tower will be a part of the regular high-service system.

The total consumption averages 2.7 m.g.d. of which 55 per cent. are supplied by gravity and 45 per cent. are pumped. The total number of miles of mains is 86.86. The total number of services is 4 869 and the number of hydrants, public and private, is 721. We are 83.15 per cent. metered, and all meters are owned and set by the city. The water is chlorinated at the rate of 3 lb. per m.g. and in their endeavor to preserve the purity of the water, the Board has since 1892 purchased property around the lake, particularly the shore line, so that now the city owns over 99 per cent. of the shore line at a total cost of $99 889. As is usual when such property is bought, a few owners asked rather high prices, but in most instances the Board was met fairly, and reasonable prices were paid. All boating on the lower part of the lake is prohibited as well as within ¼ mile of the intake. To place a boat upon the lake, it is necessary to obtain a permit from the Water Works.

The management of the works is placed in the hands of a Board consisting of eight members with the Mayor *ex officio*, and all the receipts are deposited to the credit of the Department for the payment of bonds, interest and maintenance. No charge is made for water used for municipal purposes, with the exception of the Union School District, neither is any charge made for private hydrants or sprinkler services. The total cost of the works to date is $1 625 047.73, and the bonds outstanding amount to $258 000.

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*Vol. 47, 1933, p. 1.

PUBLIC WATERS POLLUTION CONTROL IN NEW HAMPSHIRE.*

BY CHARLES D. HOWARD.†

[*Received June 6, 1936.*]

In the sense that, in the final analysis, the pollution problem in very large degree concerns the public health and safety, this is a matter appropriately for control by the State Board of Health. It is so closely bound up with the problems of public water supply, of sewage disposal, of safe bathing, of safely edible fish and of the other hygienic aspects of recreational activities — with the general control of all of which matters the health board is specifically charged by law — that any independent efforts in this direction by whatever other agencies cannot proceed far without inevitably encountering these phases of public health and safety.

It seems worth while to take this opportunity to give emphasis to this feature, because there is in evidence in a number of states at the present time, notably in certain projected federal legislation for the control of interstate waters, a tendency to assume that public waters pollution control is something appropriately to be handled by special boards or commissions, without any well defined reference to the established health agencies. As a matter of fact, these agencies are the ones, and in most cases the only ones, which have been more or less actively coping with this matter for many years.

Thus, there are, or have been, pending in this session of Congress as many as ten bills dealing with pollution control of interstate waters, and not one of which measures, as we are informed, specifically provides for the active participation of State health departments in carrying out the reforms sought to be accomplished. As contrasted to these bills, however, there is in addition a later measure, introduced at the instance of the organization of State and Territorial Health Officers, which would place such a matter under the general administration of the United States Public Health Service, with provision for the active coöperation of the various State health departments. This is the logical plan.

In this connection, it has become apparent that in view of the magnitude of the problems here involved, intrastate as well as interstate, no substantial accomplishments in this direction can be hoped for without the provision of federal aid. This, we are obliged to concede, promises the most hopeful means for attaining the ends as set forth in the resolution quoted. It is with some reluctance that this acknowledgment is made. Personally,

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*Taken from an address presented at a meeting of the New Hampshire Academy of Science, May 30, 1936, as part of a symposium on conservation of natural resources.

†Director, Division of Chemistry and Sanitation, New Hampshire State Board of Health, Concord, N.H.

I am a believer in the principle that the State should be left free to manage its own business in the way in which it sees fit. Each State, in its internal affairs, ought to be master in its own house. Unfortunately, the trend is, and for some time has been, away from this idea. With the provision of federal funds to provide for the conduct of what may be purely internal affairs, the surrender of State departmental independence threatens to become more apparent. Granting that certain advantages thereby will be gained, to me the general prospect is in some respects a disturbing one.

Nevertheless, we will have to admit that it would be idle to hope to achieve at this time radical accomplishments in New Hampshire in pollution control solely on the basis of State or local provisions. We could, perhaps, succeed in obtaining some additional legislation — although we have good reason for appreciating that even this would be by no means easy to do in so far as any mandatory requirements for sewage exclusion, for cleanups, and for provision of any State or municipal funds to this end might be involved. And toothless laws, or laws that cannot be enforced, are of but very little use. We know from experience what it means to endeavor to secure the enactment of those which are painful to our communities and industries. It is easy enough to get together and to talk about the needs of having these reforms accomplished. Their achievement, however, inevitably calls for money, and it is a well known fact that the nerve leading to the pocket-book is an extremely sensitive one. This is reflected in the rather studied exclusion of public waters pollution control from the programs of some of the regional associations for recreational development.

It will be of interest to mention briefly a few of the more important laws relating to water pollution. For the most part, these are contained in Chapter 141 of the Public Laws, which is inclusive of some thirty-three sections and the enforcement of which, with but few exceptions, devolves upon the State Board of Health. Some of these sections, enacted as long ago as 1885 and long since become antiquated, have resisted attempts at amendment. Others, fortunately, have been amended in the interests of progress, and new provisions have been introduced from time to time. It is believed that at the present time the statutory means thus provided for control of the purity of public water supplies are reasonably complete and satisfactory.

The lumber and manufacturing industries have, however, thus far been able to block all efforts at repeal of an archaic provision contained in one of the sections relating to pollution of domestic water supplies. Enacted as a concession to these interests in 1895 and based upon the fallacy of radical purification resulting from stream-flow, this provides an exemption in the case of pollution due to any of the processes of lumbering, and to any manufacturing processes where such wastes may enter "more than four miles distant from the point where the water is taken for such domestic purposes." Fortunately, this interesting exemption, so radically opposed as it is to the public good, has not in practice operated in any very serious way to obstruct our control of water supplies, as it has happened that in each individual case arising we have been able to handle the situation under other provisions.

In this connection it is interesting to note a provision enacted in 1901 which serves to prohibit the discharge into public waters of wastes from wood-working establishments set up thereafter and to require that such plants be equipped with means for excluding these wastes. Unfortunately, this prohibition and requirement does not apply to mills that were in operation prior to 1931; and there are still many such.

Boating and fishing on sources of public water supply may be regulated by the board of water commissioners, even to the extent of prohibiting these acts. Bathing therein in the case of the larger ponds and lakes, may not be done within limits fixed by the state and local health departments. In addition, the State Board of Health may, under special regulation, prohibit bathing in any part of the body of water concerned.

In the latter connection, the State Board is empowered to set up regulations as to any public water supply, forbidding any acts of contamination and the existence of any conditions or practices which might endanger the purity. Many such series of regulations have been promulgated and at the present time most of our lake and pond sources are so covered, as well as many of those involving streams. Incidentally, such regulations exist as to our largest lake (Winnipesaukee), this being considered in a broad sense a source of public water supply. While these regulations are very well observed as to individual dwellings on the lake-shore, it is a sad fact that two of our prominent resort villages continue to discharge their sewage into this lake in violation of the Board's regulations. In both instances complaints and demands for corrective action have been filed by the Board and in one of these, suit against the town was finally instituted and is still pending.

Under section 19a of this chapter, the Board is broadly authorized to require provision of any improvement in a public water supply that it may deem essential for the public good, and to order such improvement to be made, under penalty of not less than one hundred dollars for each day of failure to comply.

Under section 20, no system of water supply may be installed, or any enlargement of an existent system be made, except under plans approved by the Board. Similarly, the Board's approval must at the present time be secured in advance as to all projects for sewage disposal. This provision has a direct bearing upon pollution control in that the Board is empowered, theoretically at least, to prevent any further contributions of untreated sewage to streams, regardless of whether or not these may be polluted. In practice, in the case of a substantially polluted stream such as the Merrimack, for example, it probably would be futile, in the present state of affairs, to attempt to exact any substantial degree of purification as to any new system of sewerage. It is, however, our policy to require this as to new sewerage where the existent pollution of the water body concerned is inconsiderable.

The provision of this chapter which concerns most directly the control of public waters pollution is section 32. As amended at the last session, this now prohibits the discharge of sewage or wastes from "any dwelling, camp, factory, hotel, boarding house or other commercial establishment" into any public water "not hitherto polluted" (meaning prior to 1913), except under plans to be approved by the State Board of Health. While, in its present form, this promises a satisfactory means for keeping pollution out of many of our smaller and comparatively clean streams, it is important to understand that, as concerns existent pollution, there is no law, and the Board has no powers as to those waters which are and long have been materially contaminated,— this short of the institution of proceedings charging the existence of a public nuisance. The only way by which any really effective authority can be had for cleaning up these waters will be by act of legislature. Incidentally, as bearing upon criticisms lately voiced concerning the comparatively small number of sewage treatment plants in New Hampshire, the Board has no specific authority to require these to be installed as to existent systems; nor, for that matter, may it require a town or district to install sewerage, notwithstanding that the need of such systems as to a considerable number of places has by now become rather acute.

Contrary to ill-founded stories that have been given some currency, it is well to emphasize that, except as here noted, our ponds and lakes are clean and that our Board has ample authority for keeping them so. The need here, actually, as in the case of streams, is provision of adequate inspectional facilities. In practice we are obliged to leave oversight very largely to local health officers, many of whom are disposed to give the matter little or no attention.

Except for the two situations on Winnipesaukee, as mentioned, the only lakes receiving direct additions of sewage at this time are Winnisquam, Opechee and Mascoma. Concerning the two first-named, an attempt by legislative enactment to require the city of Laconia to install treatment failed at the last session. Just now we are trying to do something for the improvement of Mascoma. Sanitary surveys have been made by the Board of these three lakes, as well as of others, and of some of the streams. Through the interest of Ex-Governor Winant, quite elaborate surveys have been made of the Contoocook and of the Ammonoosuc Rivers. Draining as it does the heart of a most important summer playground, the extensive pollution of the Ammonoosuc by hotel wastes is to be deplored. The same may be said of Lake Winnisquam.

It is of interest to note in this connection the special legislative enactments affecting two of our streams, one of which required the Board to compel a clean-up of the Wild Cat and Glen Ellis Rivers in the town of Jackson, the other, prohibiting the deposit of solid wastes (but not sewage) in the Mascoma River. Incidentally, a fact apparently but little known is that under a special regulation of the Board, based upon the so-called nuisance law, the deposit of rubbish in any of our public waters, whether or not these may be polluted, is forbidden.

In this general connection, the matter of control of bathing in public waters is of some pertinence. Chapter 141: 7, Public Laws of New Hampshire, in effect forbids bathing in any waters constituting a public supply within such limits as the State or local board of health may fix. The significance of the word "or" here has been assumed to mean that in case the State Board chose to fix a more distant limit than did the local board, this limit would be the effective one. Furthermore, under this provision and from an analogous interpretation of the statute relative to regulations concerning boating as given by the Attorney General, if the State Board chose to prohibit bathing entirely in a given source of public water supply, it presumably could legally do so.

However, in practice we depend in this connection principally upon the provisions of section 15, which empower the Board to make any kind of a sanitary regulation for public water supplies that it may see fit. Thus, the special regulations as to quite a number of individual sources forbid bathing therein. This is true, for example, of Canobie Lake, in Salem, a body of 320 acres. From time to time we have received vigorous protests as to this prohibition, but the rule still stands.

Such larger lakes as Winnipesaukee, Sunapee, Newfound and Squam, in which bathing is extensively done, are of course special cases. No municipal supply is derived from Squam, although it is resorted to for water supply to a considerable extent by cottagers. Nevertheless, this, like the others, is covered by special "public water supply" regulations. We have advised the Squam Lake Improvement Association that this water should not be used in any case without chlorination, and this is required as to places catering to the public. Bristol's supply (chlorinated) is taken from Newfound Lake, with a bathing limit. That of Sunapee (chlorinated) is taken from the Sunapee Harbor section of Lake Sunapee, in which section bathing is taboo.

In the case of Winnipesaukee, this affords water for two public systems, that of Weirs (chlorinated) and a reserve water supply (chlorinated) for Alton. It happens that for neither of these have any definite bathing limits been promulgated. There is no practice of bathing in the neighborhood of the Weirs intake. Laconia derives its supply (chlorinated) from a connecting body (Lake Paugus), and bathing in the neighborhood of the intake is prohibited. In addition to the two municipal sources on Winnipesaukee proper, the latter is also extensively used by many cottages and camps. The Board has therefore always regarded this lake as a "public water supply" within the meaning and provisions of Chapter 141, and for many years it has had special water supply regulations thereon under authority of section 15.

Mascoma Lake proper is not a source of water supply. During July of this year the department has issued about forty notifications under the nuisance act to persons resident on the inflowing Knox River, requiring these to provide disposal systems in order to abate the contamination existent at Mascoma Beach, a popular public bathing place and cottage resort.

Manchester's Lake Massabesic supply (chlorinated) is taken from the "Back Pond" section of this lake, which is separated from the "Front Pond" by a highway isthmus penetrated by culverts. Bathing in either section is prohibited under the Board's regulations established in 1899.

A serious difficulty associated with this problem of public waters pollution control is the present lack of general public interest. There is in evidence a wide-spread skepticism concerning the need of any action along these lines. The potent argument advanced is the comparatively large expenditures which such action would entail. The industrial concerns, which now discharge their wastes into these waters with little or no hindrance, are, as a group, distinctly cold toward these reforms. The same is true of the municipalities. These are inclined to argue that the conditions in question have been existent for a very long time; things are going along pretty well as they are; why worry and go to substantial expense to effect a change? It is obvious that until this reactionary attitude can be overcome, no real accomplishment can be hoped for — at least on the basis of state or local appropriations.

At the same time, there are grounds for serious question as to the desirability of attempting to clean up such main streams as the Merrimack and the Connecticut Rivers. There are competent authorities who contend that such water courses are the legitimate repositories for sewage; that this is the "natural" method, and that so long as the pollution load does not reach the point of constituting a nuisance, there can be no valid objection to this method. Apparently, few but engineers seem to appreciate the enormous costs which carrying out of such a program would impose upon our municipalities. It is much to be doubted if the recreational advantages accruing would be sufficient to justify these costs. In any case there can be no substantial accomplishment here without provision of substantial federal coöperation.

Finally, as relating to all of the phases of this matter of public waters pollution control, inclusive of enforcement of the laws and regulations we already have, I would like to emphasize that the present means granted the State Board of Health are grossly inadequate for this purpose. Our need is not so much more laws — although more are called for — as it is more money with which to enforce those we now have. It will bear repeating that pollution control is in very large degree a public health problem. For a third of a century the State Board of Health has done all that has been done on this matter. It has fought for the enactment of laws, has set up numerous regulations, and has waged, unaided, numerous contests with those who would pollute our waters. After all, and in spite of serious limitations upon its efforts, its accomplishment through the years has been substantial.

CHANGING A TOWN FROM FLAT RATES TO METERED RATES.

BY DWIGHT L. AGNEW.*

[*Read March 19, 1936.*]

The town of Hingham is situated on the South Shore about 18 miles from Boston, in rolling country with considerable shore line and only about three miles from the town of Hull with its attractive beaches and amusement parks. The town is purely residential. The majority of our citizens commute to Boston to their businesses. In the summer there is a large influx of people who own summer homes along the shores and throughout the town. These summer houses represent about 24 per cent. of the total dwellings in the town.

In 1879, a group of Hingham citizens thought that a thriving town the size of Hingham should have a water supply. Therefore, in that year an act was passed in the Massachusetts Legislature authorizing the incorporation of a water company to be called the Hingham Water Company.

The town was very fortunate in that it had within its borders a pond called Accord Pond, with an elevation of 145 ft., which has an area of 100 acres and a capacity of approximately 518 m.g.; the water of which was clear and of excellent quality. This was to be their source of supply. The work of laying the pipes in the town was started in 1879, and water was turned into part of the town in 1880. As most of the town was situated around the harbor, it was possible for years to supply the inhabitants by gravity, but as other sections of the town and the high hills were developed it became necessary to install a pumping station on the shores of Accord Pond to augment the gravity supply by pumping during peak demands. One of the high hills which has recently been developed is supplied entirely by the pressure system which supplies the town of Hull.

The method of charging for water was by fixture rates, payable in advance, as follows: Family service $8.00, bath tub $4.00, water closet $4.00, and hose $5.00, or a total of $16.00 for a fully plumbed house. If a party desired to use a hose, an additional charge of $5.00 was made. These rates continued in force until 1918 when a slight increase was made, namely: Family service $9.50, bath tub $5.00, water closet $5.00 and hose $6.00 or a total of $19.50 for a fully plumbed house with a $6.00 charge for hose use.

On June 30, 1933, the War Department entered a complaint with the Massachusetts Department of Public Utilities as to the price charged for electricity, gas and water sold and delivered to certain units of the War Department by different electric, water and gas companies, and as the Hingham Water Company was one of these, the Massachusetts Department

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*Manager, Hingham and Cohasset Water Co., Hingham, Mass.

of Public Utilities ordered that a hearing be held on its rates. The result of this and subsequent hearings, together with the Department of Public Utilities' study of the Hingham rates, was that they concluded that a more equitable manner of charging for water by the Hingham Water Company should be established by means of meters. Therefore, on April 13, 1934, it was ordered that the Hingham Water Company begin the progressive installation of meters in the town of Hingham and that section of the town of Norwell served by the company and charge for water supplied through said meters in accordance with the following schedule and plan:
Minimum charge $9.00 per annum which allows the use of 2 800 cu. ft. per annum divided into 700 cu. ft. per quarter. For water used in excess of the allowance included in the minimum charge, the following rates shall apply:

First 7 000 cu. ft. per quarter @ $0.32 per 100 cu. ft.
Next 32 000 cu. ft. per quarter @ $0.24 per 100 cu. ft.
Over 39,000 cu. ft. per quarter @ $0.15 per 100 cu. ft.

Let me state here that the rates ordered by the Department of Public Utilities were trial rates which are subject to adjustment after completion of the metering of the town. The Company charges $1.50 for setting meters for the original installation for all year-round customers and for each installation for seasonal customers. The Company Rules and Regulations were completely revised to make them applicable to metered service. This order for meter installation only applied to the towns of Hingham and Norwell.

The number of customers at the beginning of the metering program was as follows:

Hingham Flat Rate—Seasonal	463
Hingham Flat Rate—All Year	1 602
Hingham Metered Customers	41
	2 106

The town of Hingham was zoned by the Department of Public Utilities into five districts of approximately the same number of houses, and a specified date was set for the metering of each of these districts. Zones 1 and 2 were ordered metered by June 30, 1934, which allowed the company a very short time for doing this work after the receipt of the order. Zones 1 and 2 include a large portion of the summer residences. Before metering, it was necessary to make an inspection of every house to determine the plumbing layout, outside connections, etc., and to determine whether the meter should be set in the basement or in an outside chamber. Each customer had to be interviewed and the reason for and method of installation explained to him. This, of course, took considerable time. A large number of summer residences were closed at the time the metering program began, and this necessitated getting in touch with the owners by letter or telephone. Many of these people requested that the installation be postponed until they opened their cottages. In these cases the water was shut off at the street stop, and the meter was set and the water turned on after the arrival of the summer customer. The Company absorbed the expense of setting the meters with the exceptions of those meters set in meter vaults.

Where the pipes were laid on top of the ground or just a few inches under, as was the case with a number of summer cottages, meters were set outside just before the service entered the building, and where a house was set on posts the meter was installed on the riser.

Another decision to be made was as to the type of setting to be used. Something was called for which would be economical and easy to install in small places, bearing in mind that a large number of the meters would have to be removed every fall and set every spring. After considerable study it was decided that the Ford Copperhorn setting would best suit our conditions, and that type of setting has been carried through the entire program.

The next problem was how to get these meters placed in such a short time. The local plumbers were called in and the situation explained to them, and they were asked what they would make the installations for if the Water Company furnished all material. They were not particularly anxious to do the work as it would interfere considerably with their spring work, and they would only agree to set meters in their spare time. It, therefore, became necessary for the Water Company to do this work itself. Necessary tools, cars and equipment had to be purchased and men hired to do the work. This arrangement worked out very well, and by July 1, 1934, Zones 1 and 2 were completed. The whole program of metering Hingham will be completed by July 1, 1936. The average cost of installation without meter was $4.38. In many installations where conditions were ideal, the cost was only $1.72 for material, labor and trucking. Considering the age of the piping in some of the houses and the conditions under which these meters were installed, we consider this very good.

One of the greatest difficulties was that when the Water Company men started to install a meter, the piping being so old would break further back in the cellar. Of course, this had to be repaired at our expense. Also so many pipes were filled with rust that in disturbing the pipe, rust would be carried along, plugging somewhere in the house. Sometimes it would take hours to obtain a reasonably good service again. Another serious handicap was caused by electric and telephone grounds to the water pipe, one of our men receiving a severe shock.

At first, there was considerable opposition to meters, as each person believed it would increase his rates tremendously, and as the metering progressed, stories began to be circulated about large water consumption. Whenever such a story reached us or a complaint was received, an investigation was made, and usually it was found that the customer had read the meter ten times too much or forgot to put in the decimal point or else there were leaking fixtures. Wherever possible, the Company made check readings by months of all meters to determine any unusual consumption and to allow the customers to repair any leaks or adjust their use before the meter rates actually went into effect. This was greatly appreciated by a large number of our customers.

Because meters were installed in Zones 1 and 2 so rapidly and most of the efforts of the Water Company organization were taken up with the installations, where it was determined there was a leak, some adjustments on overages were made the first quarter, but the customer was advised that no more adjustments would be made.

The houses in Hingham had no street numbers. Therefore, to identify a house the Company installed a numbered brass tag, the tag numbers tying in with the Company's service-connection numbers. These were installed on the right-hand corner of the house just above the foundation. This made it possible to identify a house for meter reading and was of considerable help.

The greatest problem, we believe, was in the billing department, as all bills on the flat rates had been sent out for the year on January 1, 1934. The metered rates in Zones 1 and 2 were effective July 1, 1934. This necessitated rebilling the metered districts from July 1, 1934 to September 30, 1934, canceling six months flat rates from July 1st to December 31st. In most cases, where customers had already paid their bills in full, a rebate was required which was either paid in cash or kept on our books as a credit against further use of water, as desired by the customer.

After the first year's meter installations, our system of meter installation, recording, billing, etc., has worked smoothly, and annoyance and inconvenience to our customers have been almost entirely eliminated, and they accept their water bills the same as they do the electric light, or gas bills.

To give an idea of some of the problems a Water Company has where houses are not occupied the year round we are herewith giving you a few facts for the year ending 1935:

Meters set in summer houses	1 168
Meters removed from summer houses	869
Houses shut off	2 259
Houses turned on	1 953

The reason for the difference in meters set and removed is that meters were left in chambers for test against freezing, and some customers remained during the full year. We have found that some meters left in pits, especially in sand or gravel soil, froze and burst during the winter of 1935–1936.

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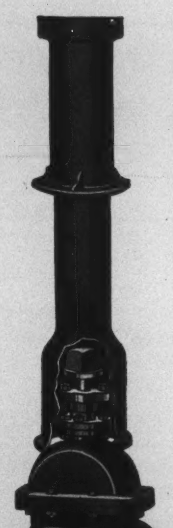

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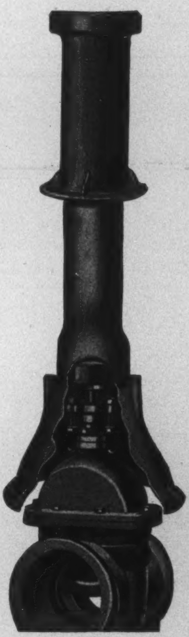

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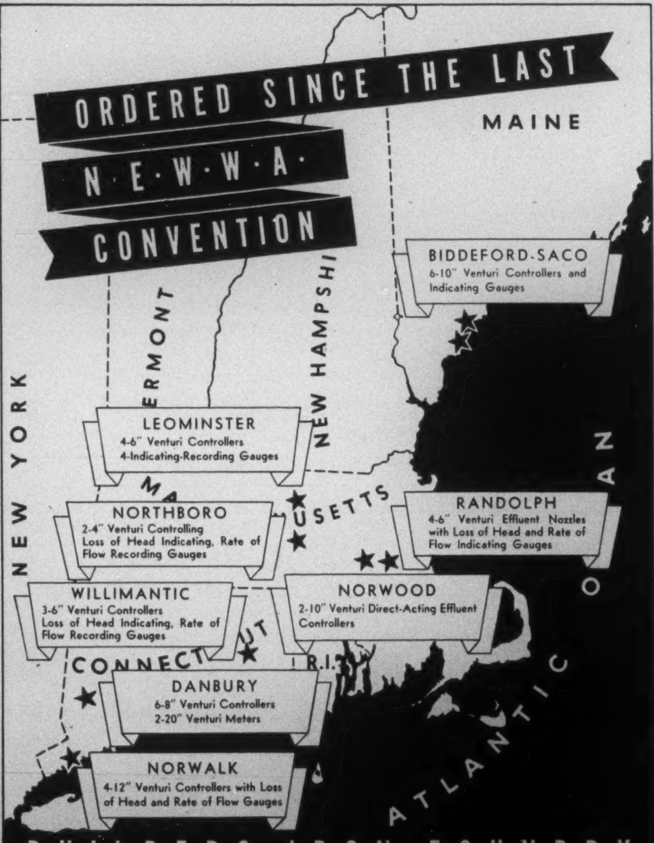

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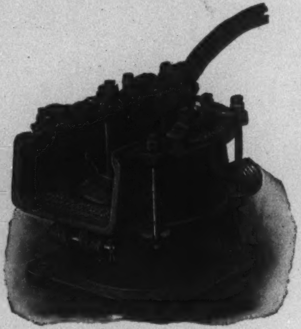

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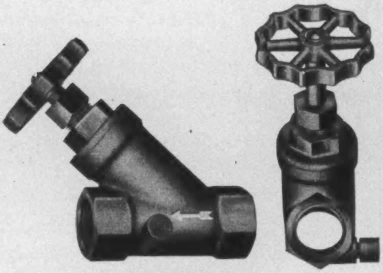

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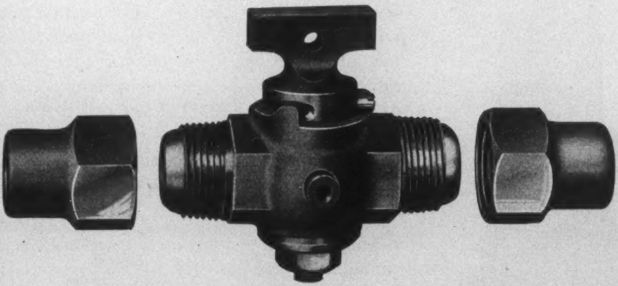

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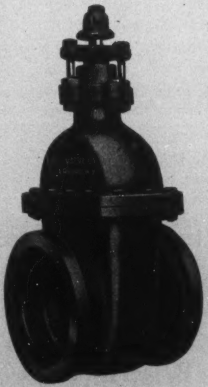

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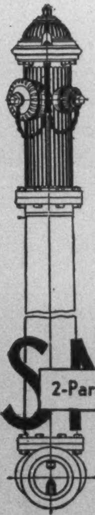

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for
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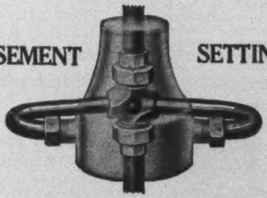

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● 2 All copper water way. No rusting or corrosion. Freezing will not burst it.

● 3 Smooth oversize tubing and sweeping bends, maximum flow.

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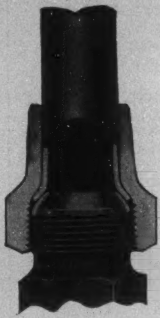

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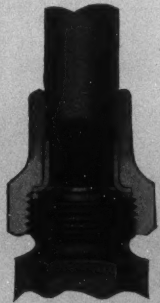

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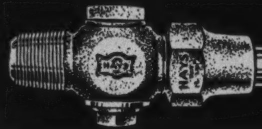

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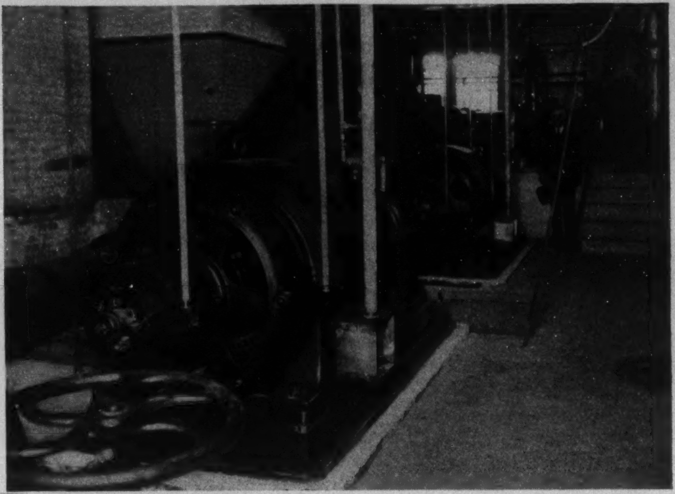

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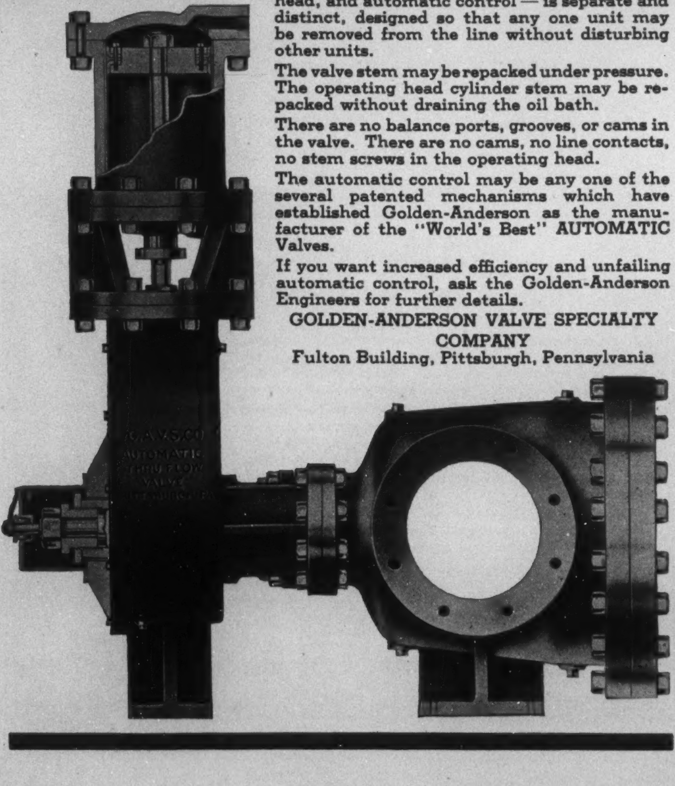

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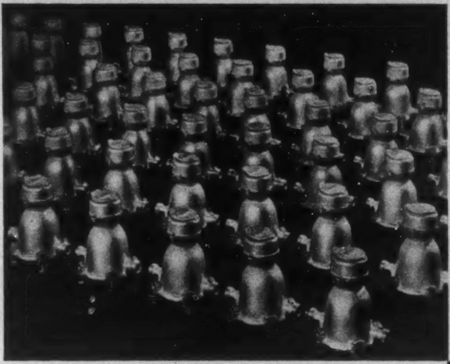

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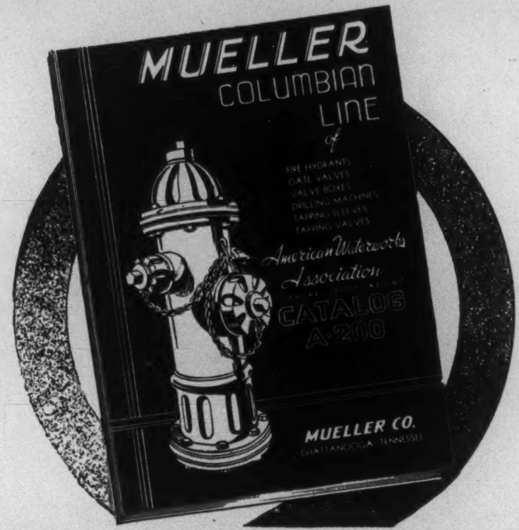
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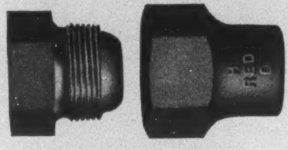

Fig. 413
Copper to Iron Coupling
Female

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Copper to Copper

Copper to Iron Male Thread

Copper to Iron Female Thread

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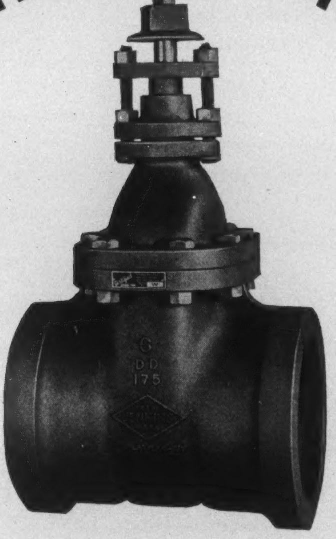

Fig. 883
Double Disc
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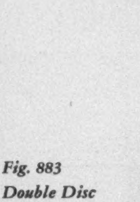

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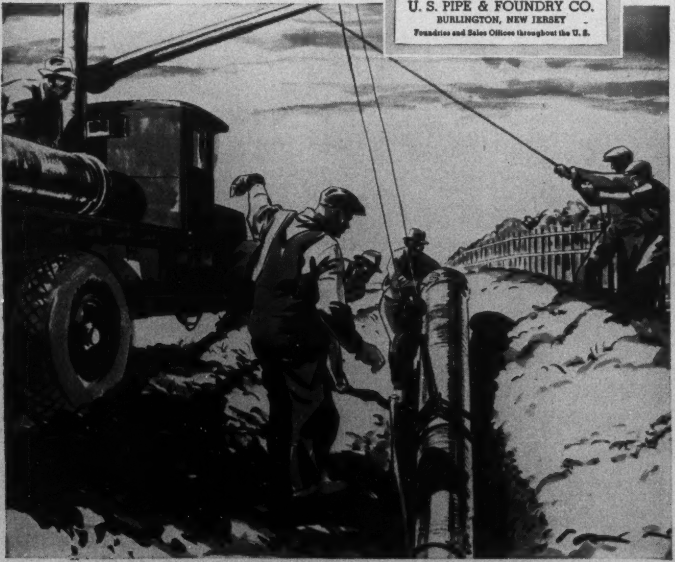

The Journal of the New England Water Works Association

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